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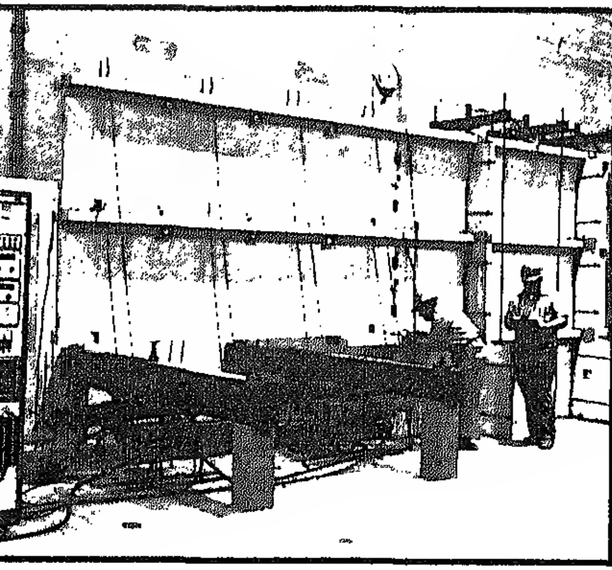

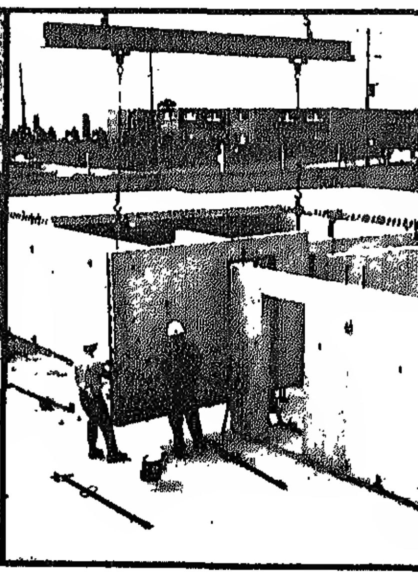

DESIGN AND CONSTRUC
LARGE-PANEL CONCRETE STRU

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This publication is based on the facts, te stated herein. It is intended for the use of pr competent to evaluate the significance and *reported findings and who will accept re* application of the material it contains. Obv Cement Association disclaims any and all r *plication of the stated principles or for* the the sources other than work performed or oped by the Association. The research and *basis for this report were conducted pursuan* the Department of Housing and Urban Devel statements and conclusions contained herei reflect the views of the U.S. Government in particular. Neither the United States nor H ranty, expressed or implied, or assumes re accuracy or completeness of the information

uctures which can affect their behavior are identified. Methods ucing the risk of progressive collapse from abnormal loads are mined. A philosophy of design based on bridging local damage is roduced which emphasizes continuity and ductility within and acro nections. A methodology is developed to be used in establishing imum detailing criteria to ensure integrity of the structure. In s General Structural Integrity approach code requirements for min s will be provided to assure an alternate path for the load in th nt of a local failure of a primary load bearing element. Design siderations for both individual elements and connections are discussed conceptually.

purpose of this report is to discuss qualita-
vely the response of the structure to both normal and abnormal lo
nditions. Specifically, the report:

1. Discusses conceptually, conditions inherent in LP structu which can effect the procedures used in the analysis and design for normal loading conditions.

2. Develops a methodology to be used in establishing minimum detailing criteria (indirect design) to ensure structural integrity of the building thus accommodating the effects abnormal loading conditions.

3. Discusses details in connections and elements which can a the behavior of the structure.

e design of each element under the applicable gravity and lateral adings should satisfy the specified conditions of "ultimate stren d serviceability. For traditional or normal loadings the stresse accounted for in design are principally limited to flexure and sl the horizontal elements and compression, flexure and shear in the tical elements. In general, methods and concepts used in the des conventional structures to resist normal loadings are applicable structures. However, to ensure satisfactory performance, the en t pay particular attention to the effect of connection details o erall behavior.

an LP structure, unless special details are employed to ensure f ntinuity at the supports, the vertical loads are carried by the f ank in a simple post and beam fashion. In the design of floor ele e engineer should consider serviceability requirements including:

ll assemblies, the simplified stiffness method is used when the wal
e solid and vertically continuous; if the walls are coupled over
penings, the shear connection, frame analysis or finite element meth
used. Under lateral loads, serviceability conditions which should
nsidered include lateral deflection and dynamic motion as it affect
ccupant comfort.

en a local failure due to an abnormal loading is not confined to th
rea of initial distress, but spreads either horizontally or vertical
hrough the structure, it is termed a progressive collapse. For abno
oads it is not the loading but the susceptibility of the building to
rogressive collapse which presents the real risk. Two modes of prog
ive collapse are defined: that caused by the inability of the struct
o form an alternate path to bridge local damage, and that caused by
nsurmountable debris loading.

hree possible approaches can be employed to reduce the risk of progr
ive collapse: eliminate the hazards which cause local failure; desig
he structure so that the hazard does not cause any local failure; or
llow the local failure to occur but ensure an alternate path for the
oad. Since the nature and magnitude of most abnormal loads are unpr
ictable, the third approach is advocated. When a structure has the
bility to bridge over local failure it has what is termed General
tructural Integrity, the principal elements of which are continuity
uctility of members, of connections, and of the structure as a whol
his additional continuity and ductility is not usually required to
esist normal loads, but allows the structure to mobilize the reserv
trength needed to resist progressive collapse.

inimum detailing is predicated upon:

1. Extent of damage -- an assessment of the nature and exte local damage that is likely to occur under abnormal load

2. Alternate paths -- evaluation of alternate structural ac which can develop as a consequence of a local failure to establish load flow in the remaining undamaged structure stability analysis of the partially damaged structure.

ased on this procedure a rationale for horizontal and vertical ti stablished:

1. Transverse ties to develop cantilever and beam action in panels;

2. Longitudinal ties to develop partial membrane action in elements;

3. Vertical ties to develop suspension action in wall panel to ensure adequate shear capacity in the horizontal conn between panels; and

4. Peripheral ties to develop diaphragm action.

The combination of system continuity and ductility should enable t structure to absorb the abnormal load with minimal damage, or alte tively to bridge localized damage as a result of the abnormal loa Thus provision of General Structural Integrity eliminates the nee

any part of every building.

In LP structures, as in any precast building, allowance must be made the design for dimensional deviations. To reduce the need for onsite modification and adjustment the engineer should adopt and specify a "tolerance" system. A tolerance system is suggested in the report, based on a survey of several systems used in North America.

In the design of wall panels the engineer should consider the followi load and geometric eccentricities, slenderness, thermal effects, and effects of connections and openings. Similarly in floor elements, th effects of wall clamping on simply supported plank, horizontal shear composite systems and end bearing are among the more critical factors which should be examined.

By its very nature, an LP building is only as strong as the connectio between the individual elements. If sufficient strength and ductilit is not developed in the connection, the available strength in the adjoining elements may not be fully utilized. Within the report, strength and performance requirements for the most widely used connection details are discussed to provide a basis for analysis and design

Connections are classified with respect to location, direction, and function. The primary connections discussed are: interior horizontal wall-to-floor; exterior horizontal wall-to-floor; horizontal floor-to floor; and vertical wall-to-wall. Variables affecting connection behavior include: precast and cast-in-place concrete properties, floo penetration, load eccentricities, floor moment and mortar packing. T function and influence of each connection on the overall behavior of structure is discussed.

conventional cast-in-place structural wall (egg crate) construct
Large panel buildings are differentiated by the general arrangem
(Fig. 2) of load-bearing walls:

(a) Cross wall system: in this most prevalent form, the l
bearing cross walls are perpendicular to the longitudi
of the building.

(b) Spine wall system: the load-bearing walls are paralle
longitudinal axis of the structure.

(c) Mixed systems: a combination of cross wall and spine
systems.

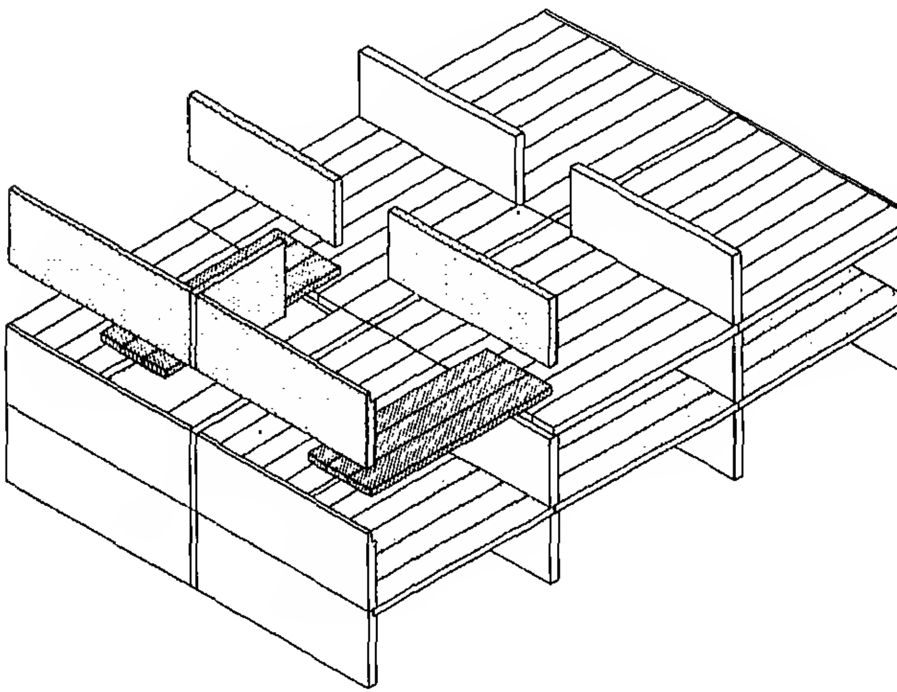


Fig. 1 Isometric View of Typical Large Panel Structure

(a) Cross Wall System (b) Spine Wall System

Fig. 2 Typical Arrangement of Structural Wall Panels in Large
Structures

n most LP systems, the walls transfer their loads directly to the ubstructure without an intermediate frame. This form of construction estricts open plans at any level and is thus most typically suited fo ultistory housing where walls of substance usually have to be provide etween apartments to resist fire and noise transmission. Constructio ypes considered under this investigative program include solid, sand- ich, ribbed, hollow core or composite wall panels; and solid, hollow ore, or ribbed floor units with or without cast-in-place topping. Al lements can be either prestressed or conventionally reinforced.

he overall program objective is to develop minimum criteria for the esign and construction of large panel structures. These criteria are eing developed to ensure the structural safety and serviceability of esidential buildings, while also providing minimum performance requir ents to be used by designers and developers of innovative systems. T evelopment of the criteria will also expand the knowledge of design a onstruction of large panel structures to a level comparable with that xisting for conventional cast-in-place concrete or steel structural ystems.

undamental requirements: safety and serviceability. Safety
ssed by ultimate strength, and serviceability as expressed in
ctions, cracking and vibrations, should be considered not only for
tructure as a whole, but also for its individual components,
ons and materials.

scussed in Report 1[1] all loadings may be classified under two
categories:

1. Normal loading: a loading condition included in the typical analysis and design of a building, following current codes and practices; and

2. Abnormal loading: a loading condition (usually of high localized intensity) which is not explicitly included within current codes and practices.

of the loads to which a building may be subjected during its life
robabilistic in nature.[1-5] Consequently, it would be desirable
rform theoretical safety analyses on the basis of probabilistic
pts.[6-9] Currently much work is being carried out on probabi-
c methods.[10-13] However, practical application of probabilistic
pts to structural design does not seem imminent. The practicing
eer is not yet expected to use such methods in daily analyses and
ns; rather, the conventional deterministic and semi-probabilistic
ngs introduced in today's codes and standards are used.[14-20]

to the introduction of probabilistic methods which would replace
eterministic values, it is essential that <u>all</u> possible loading
tions be considered to some degree.[6,8] In the case of codes and
ards for conventional steel and cast-in-place concrete buildings,

basis for evaluation (abnormal loadings) are handled indirectly thr
minimum detailing criteria for continuity and ductility of individu
elements and their connections, and through the choice of safety fa
for strength. Thus, indirect code provisions attempt to establish
adequate response of the structure to abnormal loadings. This indi
approach in traditional structures usually yielded an inherent abil
to cope with abnormal loading conditions. In effect, a minimum lev
integrity was established in most of our conventional structures.

This report is the second in a series of reports leading to the dev
ment of a "Recommended Practice for the Design and Construction of
Structures." The purpose of this report is to discuss qualitativel
response of the structure to both normal and abnormal loading condi
Specifically this report will:

1. Discuss conceptually, conditions particular to LP structu which can affect the procedures used in the analysis and design for normal loading conditions.

2. Develop a methodology to be used in establishing minimum detailing criteria (indirect design) to ensure structural integrity of the building thus accommodating the effects abnormal loading conditions.

3. Discuss details in connections and elements which can aff the behavior of the structure.

- live load,
- soil and hydrostatic pressures,
- snow load,
- earthquake load, and
- wind load (excluding tornado).

ost building codes require consideration of volume changes due to emperature, shrinkage and creep. However, these are not routinely nsidered by the engineer, due to a lack of commonly accepted desi rocedures.

s noted in Report 1, the characteristics and magnitude of codifiec rmal loading conditions are directly applicable to LP constructic scussion of response of LP structures to the above loadings, and opropriate methods of analysis and design follow. Excluded from t iscussion are "earthquake load" and "effects of volume changes", e f which will be considered separately within subsequent reports.

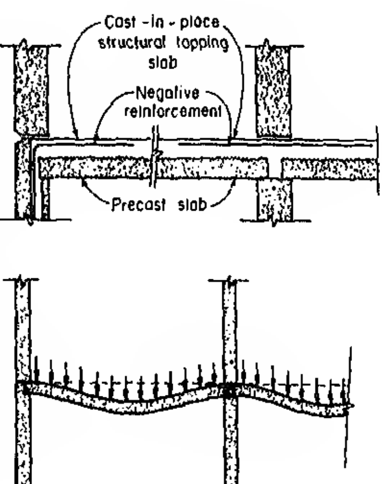
P buildings are bearing-wall structures. Such structures, consist f interconnected walls and slabs, are laterally stiff and can be orced without difficulty to resist normal loading conditions. An ssessment of the nature of the traditional loads to be discussed his section indicates two distinct categories of loadings: vertica ravity loads) and horizontal or lateral loads. The design of each ement under the applicable gravity and lateral loadings should sa he specified conditions of "ultimate strength" and serviceability.

nder vertical and lateral loads the stresses to be accounted for esign within the individual elements are principally limited to fl nd shear in the horizontal elements, and compression, flexure and

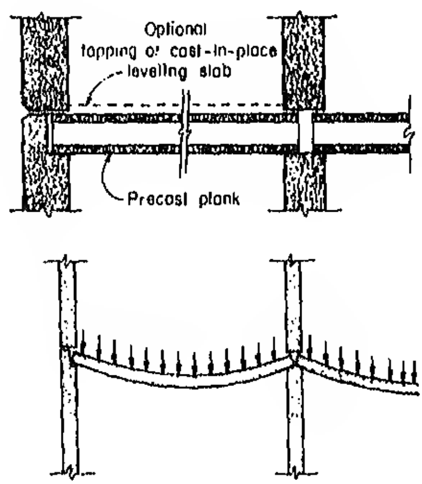
nction with an elastic analysis to determine design forces and defor-
tions.

2.2.1 Resistance to Vertical Loads

In conventional cast-in-place concrete slab systems it is customa to utilize flexural continuity at the supports in resisting gravi loads. However, an LP building subjected to such loads cannot develop flexural continuity at the supports unless a cast-in-plac topping slab is employed, or reinforcing steel loops protrude fro the slabs and walls to interlace within joints. In the case of topping slabs, sufficient thickness to accommodate the reinforcin and provide adequate bond to the precast floor elements is requir to develop negative moment continuity (Fig. 3a). Where the struc tural cast-in-place topping is omitted and no interlacing loops a used, gravity loads are carried in a simple post-and-beam fashio the slabs being simply supported (Fig. 3b).



Continuous Composite



(b) Simple Span Non-Composi

In the prestressed plank typically used in LP construction, crack control is not usually a problem; however, deflection can be a critical consideration. Computations must take into account the deflections due to both immediate and long-term effects. Slab vibrations at relatively high span-to-depth ratios should also be investigated. The short-term, immediate deflections should be calculated using the moment of inertia of the gross (uncracked) concrete section. The additional long-term shrinkage, creep and steel relaxation deflections of prestressed concrete members shou be computed for the stress level due to sustained loads. The tot allowable deflection should not exceed the limits stipulated in A 318-71.(15)

2.2.2 Analysis Techniques for Vertical Loads

Current criteria, formulae, and specifications contained within codes(14-20) and related literature(21-27) for the investigation strength and serviceability are based on established analytical techniques. Consequently, a general discussion of gravity load resistance of LP buildings is not included here. However, specif design details pertaining to LP construction which may affect the strength and serviceability of the completed building will be discussed in detail within subsequent sections of this report.

2.2.3 Resistance to Lateral Loads

Continuous vertical wall elements in both the transverse and longitudinal direction of LP buildings are the basic structural elements to resist lateral loads. Although frame action is feasi ble and so et es used in one of the orthogonal directions of the

related to the center of rigidity of the entire structure must be considered when distributing loads to these elements. Where the center of rigidity of the resisting elements and the center of mass of the structure do not coincide, torsion will be introduced into the structure. Because torsion due to lateral loads can produce significant stresses, particularly in peripheral elements, a more or less symmetrical arrangement of the principal lateral load resisting elements should always be attempted. If torsion due to assymmetry is unavoidable, then the analysis and design should reflect this condition.

In addition to strength requirements, the following serviceability conditions should be considered:

(a) lateral deflection of the structure, particularly as this may affect the stability and cracking of structural and nonstructural elements; and

(b) motion of the structure as it may affect the comfort of occupants.

Lateral deflection or drift is the magnitude of displacement at the top of a building relative to its base. The ratio of the lateral deflection to the building height is referred to as the "deflection index". Maximum allowable drift is imposed to limit possible adverse effects on the stability of individual wall elements as well as the structure as a whole, and to preserve the integrity of nonstructural partitions, glazing, and mechanical elements in the building. No systematic study has yet been published to determine the precise relationship between drift and the above factors.

To date only the Uniform Building Code[15] and the National Building Code of Canada[20] among North American model building codes

...some reinforced concrete buildings which ... or exceed this criterion appears to have been satisfactory with respect to the stability of the individual elements and the structure as a whole. Recent studies have indicated that for tall concrete buildings containing structural walls*, computed deflection indices ranging from 1/800 to 1/1200 are common and easily attainable.(30,31) For LP buildings constructed with structural wall elements in both directions, it would appear reasonable to expect a deflection index of at least 1/1000.

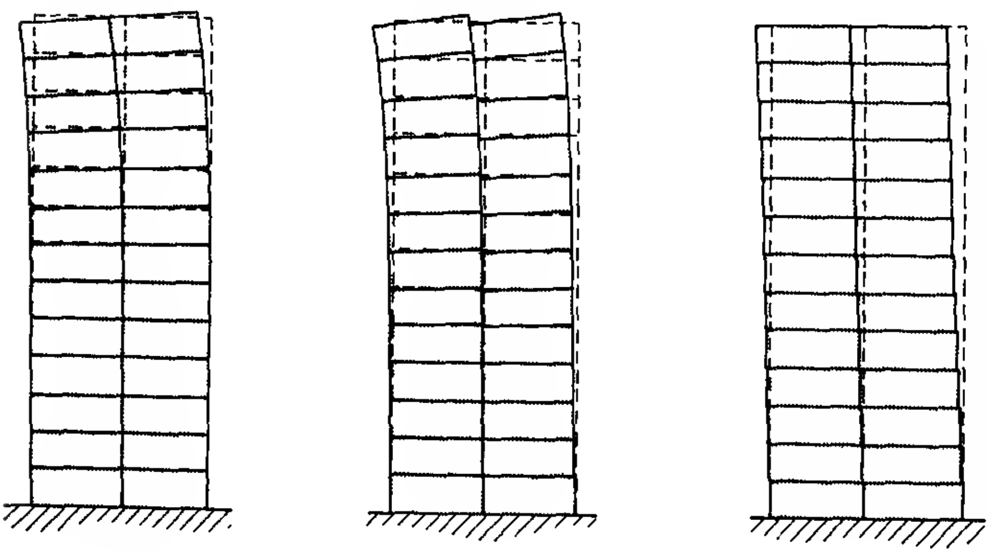
The sway motion of a tall building under turbulent wind, if perceptible, can produce psychological effects which render the building undesirable from the user's viewpoint. The sensation of motion which can be disturbing to an occupant of a building may result either from the visual perception of a relative displacement, or from the acceleration of the floor on which the observer stands. It has been noted(30,31) that no perceptible motion has been reported in residential, shear wall <u>concrete</u> buildings to date. This is apparently attributable to the high rigidity (and associated low deflection index--1/1000) inherent in tall structural wall buildings. Again, considering that LP buildings are quite stiff, there is little likelihood of detrimental effects ... way.

2.2.4 Analysis Techniques for Lateral Loads

In an overall analysis of the lateral resistance of an LP structure, certain assumptions must be made about the effect of connections. Wall assemblies in LP buildings, consisting of story-high wall elements and horizontal and vertical connections, are non-homogeneous in nature. Therefore, to determine the strength and stiffness of a wall assembly, the ... of joint characteristics

working stress levels. In calculating the increased shear resistance of the horizontal connections due to vertical load, allowance should be made for the effects of sustained gravity loads only.

A wall unit may be a solid wall composed of single or multiple panels or a series of separate walls connected either nonrigidly or rigidly by beams. In nonrigid linkage, the connection beams are usually within the thickness of the floor construction with openings equal to a story height. Rigid linkage is usually created by portions of the wall, thus openings are less than story height. Because of high bending and shear stresses, careful attention is usually given to the design of such connecting beams.



(a)

Monolithic

(b)

Non-rigid Vertical Connections

(c)

Non-rigid Horizont[al] Connections

. 4 Response of Panel Wall Assemblies under Lateral Loads Based on Connection Characteristics

openings involve simple bending theory only. Similarly, in struc-
tures whose walls are nonrigidly linked together with lintels
incapable of moment transfer (typically at corridors), the later
loads are distributed on the basis of relative stiffness (Fig. 6

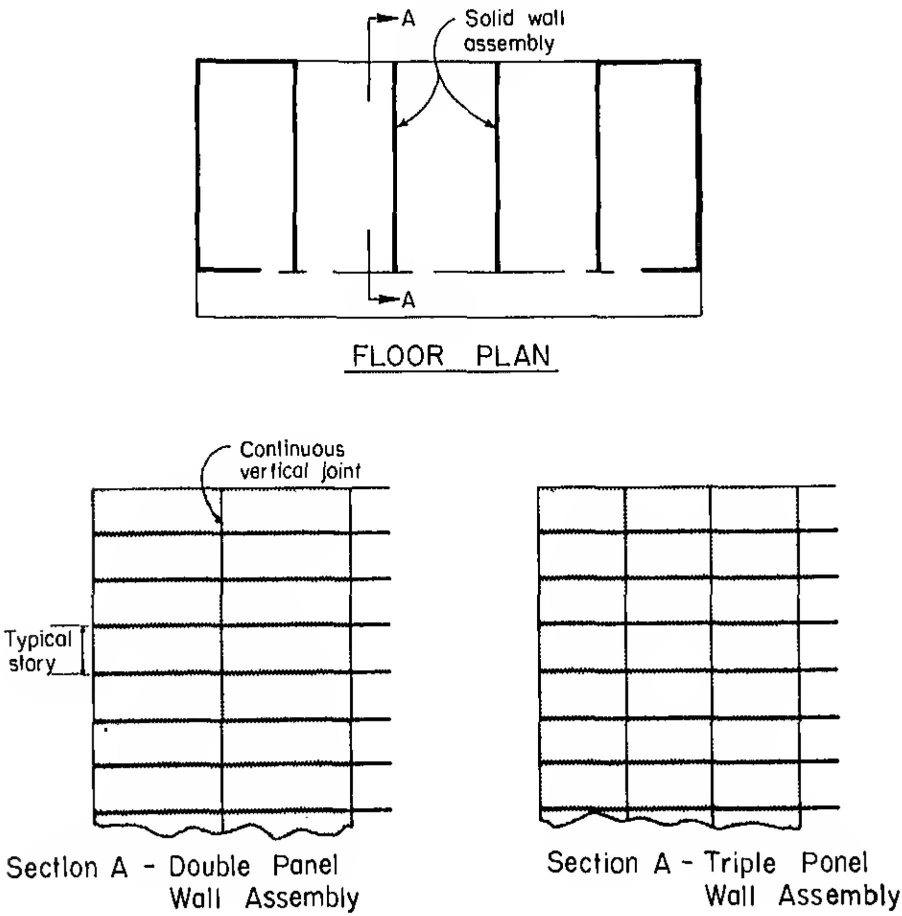
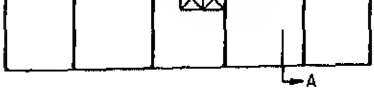
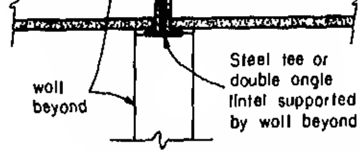
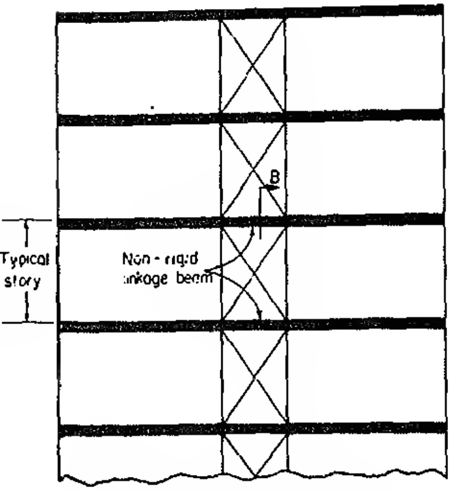


Fig. 5 Solid Wall Assembly





Section B - Non-Rigid Link Beam In Floor System with or without Topping



Section A - Double Panel Linked Wall Assembly

Fig. 6 Linked Wall Assembly

The distribution of lateral load for the systems described ab
based on the assumption that floors remain rigid within thei
planes, so that each wall assumes the same deflected shape.
uniformly distributed lateral load, "W", the loading is dist
in the proportion:

$$\frac{I_1}{\Sigma I} W : \frac{I_2}{\Sigma I} W : \frac{I_3}{\Sigma I} W : \ . \ . \ .$$

to each wall. I_i is the moment of inertia of the wall, i, a
$= I_1 + I_2 + I_3 + \ . \ . \ .$ (shear deformation is neglected).

analysis must take this factor into account.[32-34]

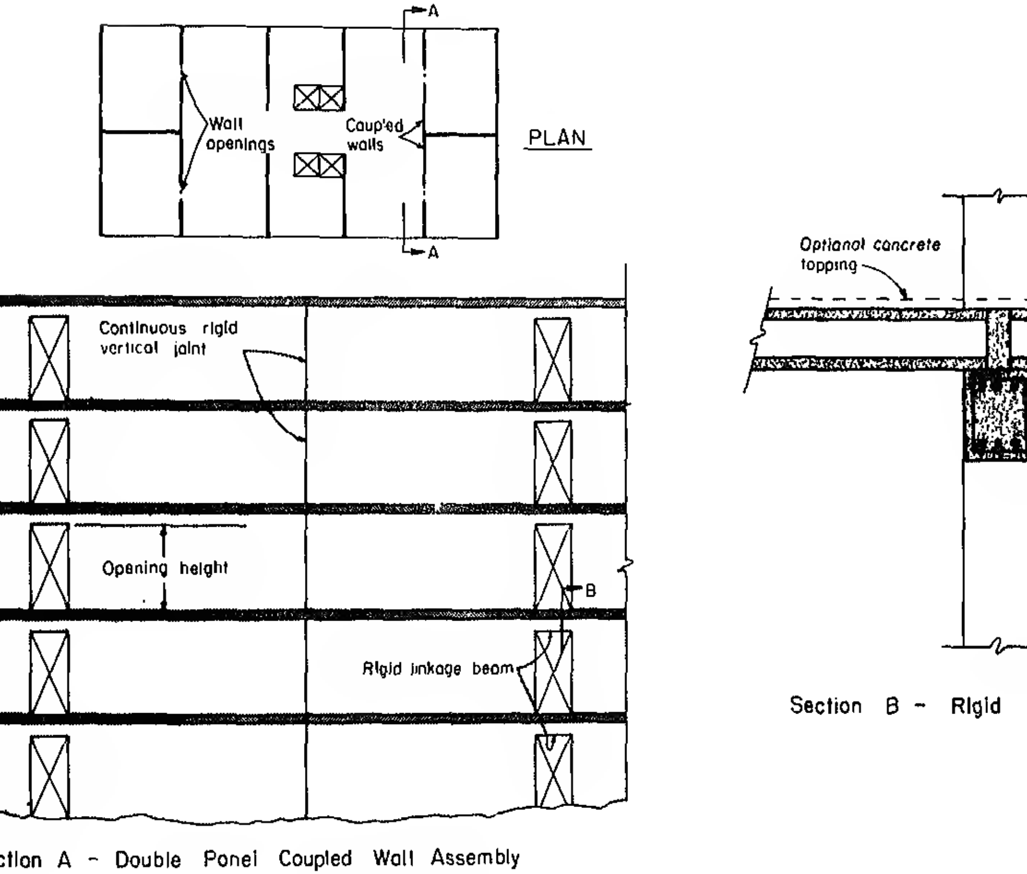


Fig. 7 Coupled Wall Assembly

The proportion of the total lateral load which each coupled wa assembly resists can be determined by inserting the deflectio reciprocals in place of the moments of inertia in Eq. 2.1:

$$\frac{1/\delta_1}{\Sigma 1/\delta} W : \frac{1/\delta_2}{\Sigma 1/\delta} W : \frac{1/\delta_3}{\Sigma 1/\delta} W : \ . \ . \ . \quad ($$

Openings in wall units generally occur in vertical rows (Fig. Typically, the connection with the wall panel is provided by a reinforced beam which forms part of the wall unit itself (Fig. Sec. B). The terms "coupled shear walls", "pierced shear wall and "shear wall with openings" are used to describe such wall assemblies.

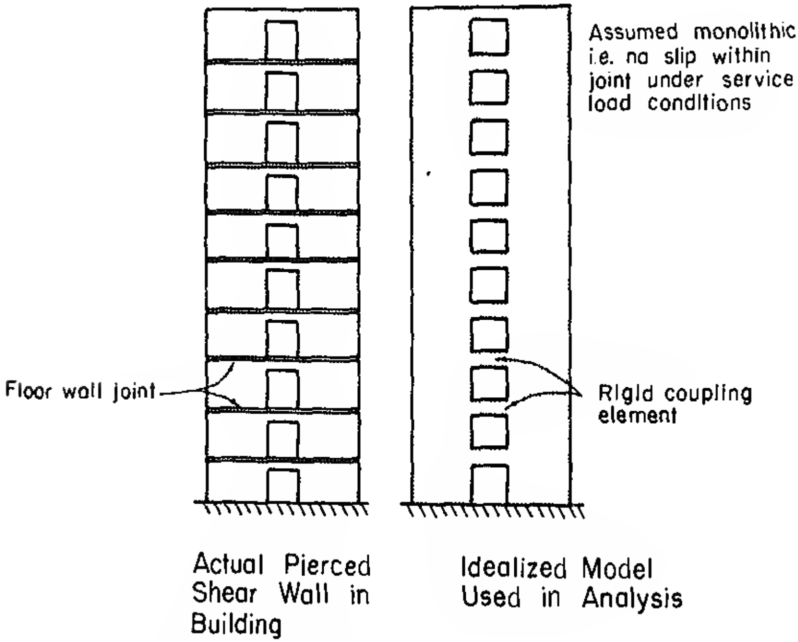
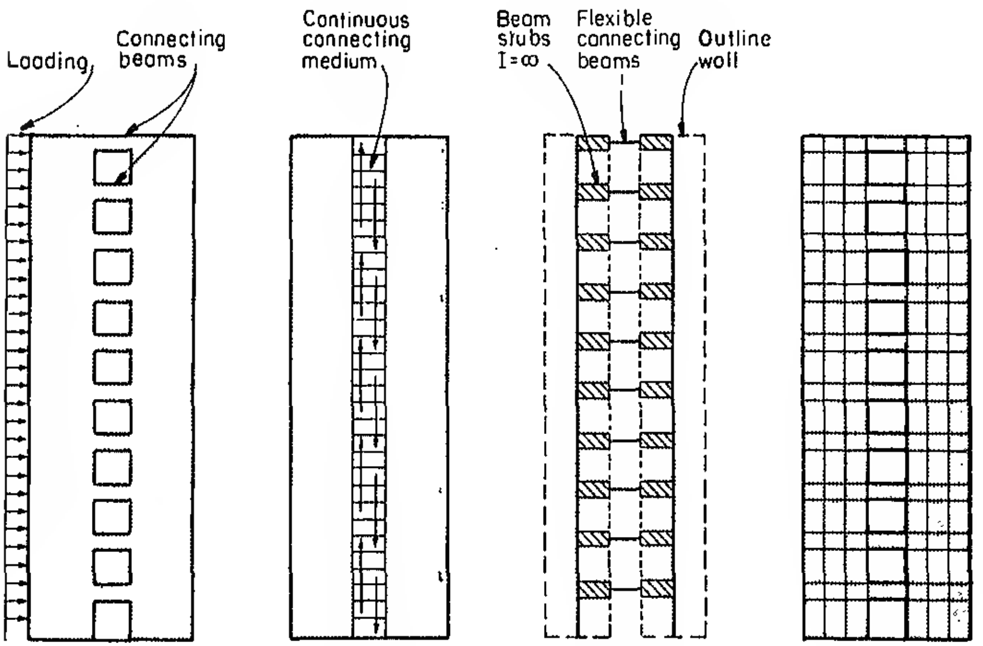


Fig. 8 Idealized Coupled Wall Assembly

If the openings are small (such as windows), their effect in a structural wall may be minor. Larger openings have a more pronounced effect and can result in a system in which frame ac predominates. The degree of coupling between two walls has bee expressed in terms of a geometric parameter, α, having a unit 1/length, which gives a measure of the relative stiffness of t connecting beams with respect to t at of the wa

separate cantilevers. For all intermediate values of αH, the stiffness of the connecting beam should be considered.

Prior to the early 1960's, no practical analytical techniques for coupled walls were available. Consequently, coupled walls were designed as individual cantilevers, each carrying lateral load in proportion to its stiffness (moment of inertia). Although these structural walls were substantially overdesigned, the approach resulted in underdesigned connecting elements. Since the 1960's, considerable amount of information has become available(35-43) an a number of practical approaches have been suggested. Reference presents a detailed comparison of the various analytical approach



(a) Coupled Wall (b) Shear Connection (c) Frame Analysis (d) Finite Element

Fig. 9 Analytical Techniques for Coupled Walls

is based on a second order differential equation which yield
deflection, moment and shear.

Possible sources of inaccuracies in the method are: local wall deformation,[44] greatly differing wall stiffnesses,[45] and reduction in stiffness of coupling elements due to cracking.[46] While the method has certain limitations, the shear connection method can be used to investigate the basic behavior of coupled wall systems.

Frame Analysis Method--Analytical methods employing computers are more accurate and considerably more flexible as they can take into account many more variables than the shear connection method. A g computer technique analyzes a coupled wall as a frame, recognizing the finite width of the columns in comparison with the beams.[47-4 Infinitely stiff beam stubs from the centerline of the wall to the edges of the actual opening are introduced as illustrated in Fig. 9c. This method takes into account changes in wall thickness, story height, and concrete strength within the height of the building.

Finite Element Method--The technique known as finite element analysis considers the wall to be divided into a mesh of two-dimensional elements (Fig. 9d). By imposing the appropriate boundary conditions a solution can be obtained by matrix techniqu which involve the solution of many simultaneous equations. The accuracy depends on the fineness of the mesh and the sophisticati of the finite element, each of which in turn affects the computer running time.

made to date.

2.2.5 Tensile Stresses

As a rule, economic considerations dictate that the walls of larg panel buildings be designed with as little reinforcing as possibl Wall panels in the past have varied from totally unreinforced in lower structures which are less susceptible to unusual risks,[51, to moderately reinforced in taller buildings, especially in seism areas.[51,53-56] Conflicting minimum reinforcement requirements have been suggested in a number of recommended practices[53,57] a building codes.[15,58,59] These vary from zero to a value of 0.2 of the gross area of concrete, both vertically and horizontally. Since wall panel reinforcement is considered in detail in the subsequent report on walls, the discussion here will be limited to minimum reinforcement requirements for the completed structure.

In tall structures tensile stresses often develop in wall assembl under lateral load conditions; net tensile stresses due to bendin can also occur. As a result, it is necessary to ensure tensile continuity between consecutive wall members. The building code requirements for structural plain concrete[58] allow net tensile stresses on the cross section of unreinforced concrete members. This is a safe specification only if the homogeneity of the concrete elements is assured throughout the structure. In a high-ri LP building it does not seem reasonable to assume that all wall panel sections will remain uncracked during fabrication, erection and the full service life of the structure. In addition, the connections at the floor level cannot transfer any tensile forces without reinforcing crossing the joints. Accordingly, it appears consistent with current practice to require that all net tensile

(Fig. 10):

(a) compressive and tensile stresses at the longitudinal boundaries of the diaphragm,

(b) shear stresses at the transverse boundaries of the diaphragm, and

(c) shear stresses along the longitudinal and transverse edges of the floor plank.

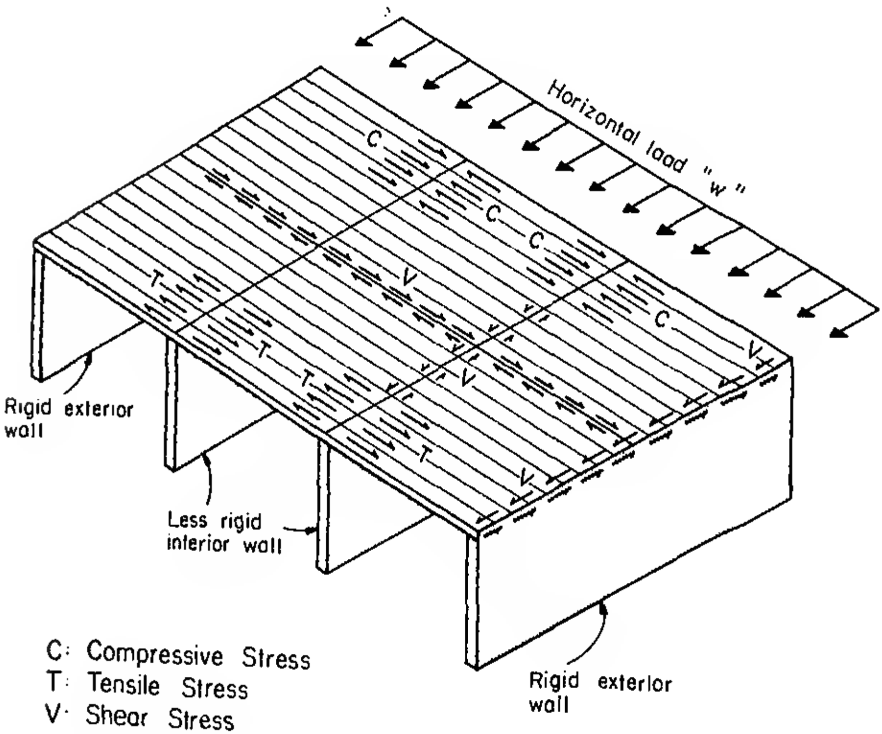


Fig. 10 Stresses in Horizontal Connections induced by Diaphragm

in combination with, the grouted keyways, mechanical connectors m
be used. Alternatively, a structural cast-in-place topping properly bonded to the precast plank can act as a diaphragm.

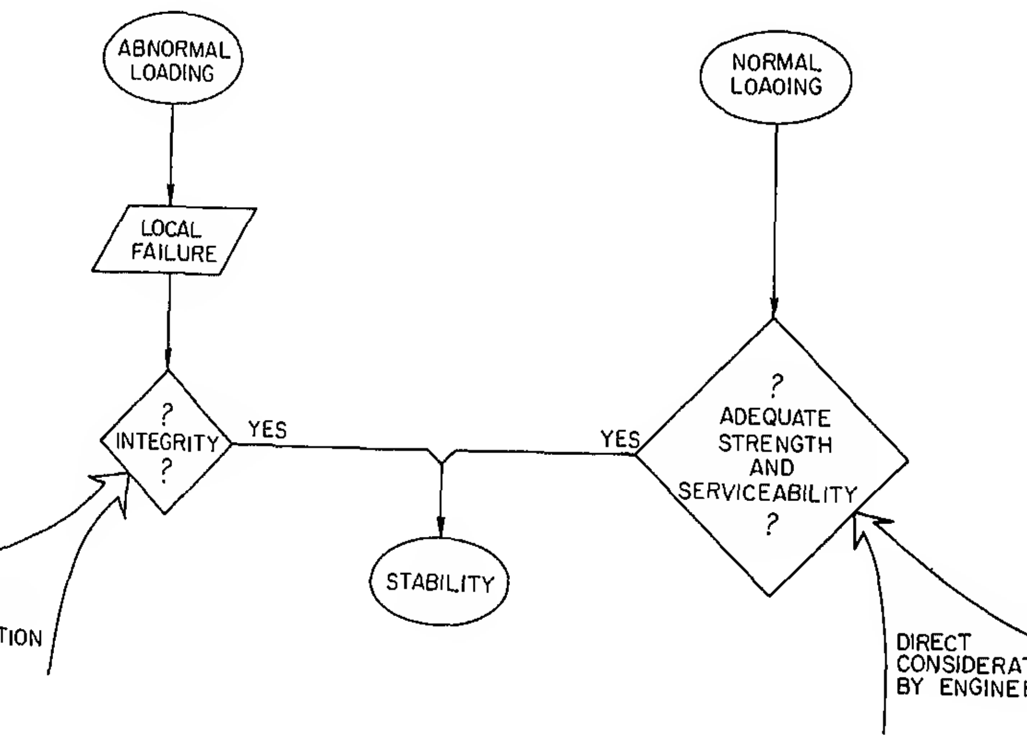
2.2.7 Concluding Remarks

With few exceptions, methods and concepts used in the design of conventional structures are applicable to LP construction. Howev
to assure satisfactory performance in relation to both strength a
serviceability, the structural engineer must pay particular atten
to the effect of connection details on overall behavior.

Except under earthquake loading and severe wind loading, numerica
analyses generally do not indicate a need for tying the component
together. As a consequence, joints could be detailed with little or no continuity provided between members. The resulting structu
would depend solely on friction and gravity for its stability. Such a structure, although designed for all code-specified loads, may, in fact, be extremely susceptible to total or partial collap
from a local failure caused by an abnormal loading condition. To assure the stability of the building under <u>all</u> loading conditions that may occur during its service life, the response of the build
to abnormal loading conditions must also be considered.

phenomenon in which the spread of a local failure eventually results in the collapse of an entire building, or a disproportionately large part of it. Abnormal loading conditions and effects which can be structurally significant[1,61,62] include:

- service system (gas) explosions,
- explosive bombings,
- external explosions from accidents involving transportation of hazardous materials,
- ground vehicular collisions with buildings,
- aircraft collisions with buildings,
- tornados,
- flooding,
- foundation settlements, and
- errors in design and construction.



11 Schematic Diagram of Design Pr cess with Adequate Detailing

structural integrity which would provide a certain resistance to fects of abnormal loadings (Fig. 11). Ductility of members and co ons would help to redistribute loads from overstressed sections. treme cases when local failure occurred, continuity between member uld help provide an alternate path for the load. These beliefs we stified for some traditional forms of construction which have con- nuity and ductility; however, they were not valid for every form c nstruction.(66-74)

1968, attention of the engineering community was focused on the oblem of lack of structural integrity in certain precast structure tragic way. In May of that year a progressive collapse of an apar nt building occurred at Ronan Point in London (Figs. 12,13). A ga plosion in an apartment on the 18th floor caused an exterior panel blown out; this initiated a progressive collapse upwards to the r d then almost down to the ground as debris fell on succeeding floo llowing the partial collapse of this large panel building, a publi quiry was held and a report describing the disaster was issued.(75 e inquiry determined that the design complied with all applicable ilding codes and that there were no deficiencies in workmanship. s further determined that within the structural system, some eleme sisted lateral forces solely by bond and friction.

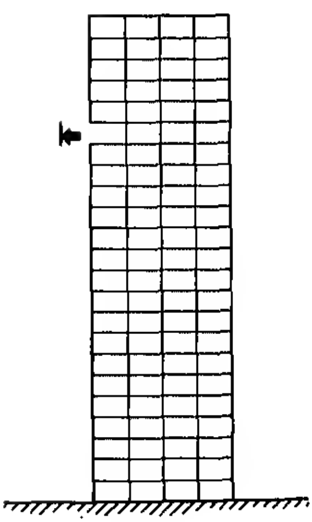
is incident prompted further investigations into other collapses roughout the world.(76-78) These and other studies indicated that oblems associated with progressive collapse and abnormal loads cor rned multistory construction of all forms, and were not limited rictly to large panel construction.(61,64,72-74,79-87) As a resul ese investigations, two facts became evident:

Fig. 13 Collapse at Ronan Poin

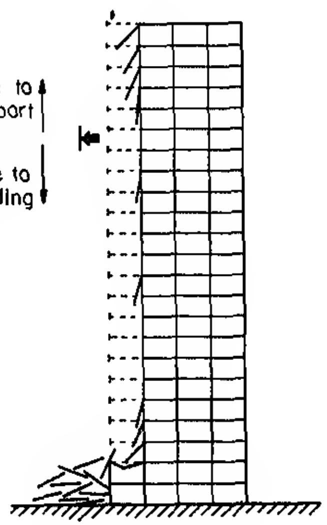
2. Resulting from insurmountable debris loading (e.g. th
of Ronan Point from the 18th story to the ground).

2.3.1 Reducing the Risk of Progressive Collapse

The explosion at Ronan Point initially caused only local d
that is, a wall element was blown out from the structure (
14a). In terms of hazard, the initial damage was of minor
sequence. The progressive collapse which occurred (Fig. 1
however, was the result of the inability of the structure
over the local failure, that is, its lack of integrity. T
it is not the hazard of abnormal loading, but the suscepti
a structure to progressive collapse which presents the rea
(Fig. 15).



(a) Immediate Local Damage



(b) Progressive Collapse

Fig. 14 Failure Modes of Ronan Point Collapse
(form Ref. 86c)

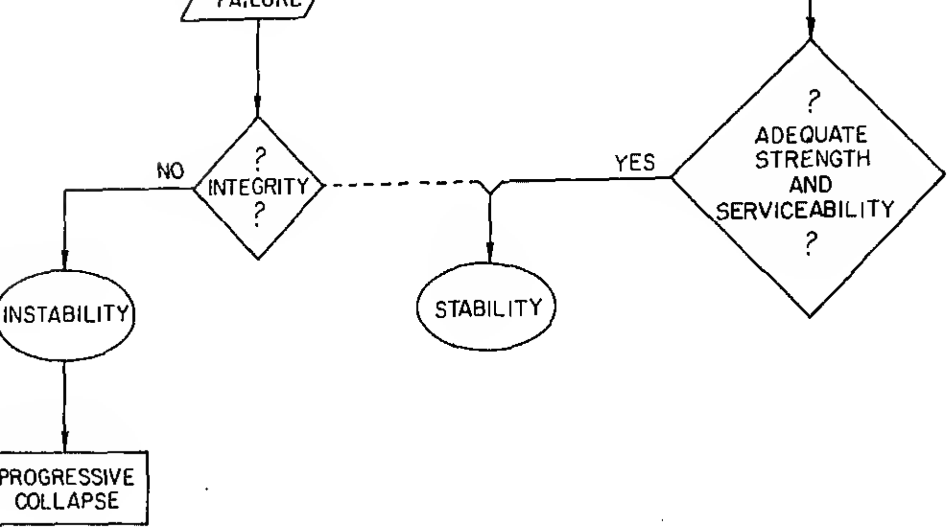


Fig. 15 Schematic Diagram of Design Process with Inadequate Detail Standards

Three possible approaches widely varying in concept can be employed to reduce the risk of progressive collapse:[88]

1. Eliminate the hazards which cause local failures (e.g., elimination of gas installations in multistory buildings, as has been done in France).

2. Design the structure so that the hazard does not cause any local failure (e.g., 5 psi gas explosion load requirement initiated in Great Britain).

3. Allow the local failure to occur but design the structure so that progressive collapse does not occur (ensure an alternate path for the load).

provision of venting to relieve explosions, use of shock absorbers for crash barriers against vehicular collision, prohibiting construction in flood plains, etc. In this way, the forces and their effects due to certain abnormal events can be reduced or eliminated, thus lowering the overall risk from abnormal events. With the above exceptions, however, abnormal loads can hardly be eliminated or controlled. Sinc any realistic solution must deal with all abnormal loading conditions to some extent, this method of eliminating the hazards is not considered an overall or complete solution.

2.3.1.2 Local Resistance

In the second approach, providing local resistance, prescrip- tive loading requirements are stipulated in an effort to provide sufficient strength to resist a local failure. A logical approach for dealing with the infrequent occurrence an abnormal loading condition would be the probabilistic method.[65,89,90] Although in theory the probabilistic meth may provide the answer, many elements necessary for its development are still lacking. Once suitable probabilistic design codes are developed and implemented, abnormal loads will become a part of the overall safety considerations. However, implementation does not appear to be imminent.

In lieu of a probabilistic approach, a minimum design load o 5 psi for gas explosions could be used. However, this appro which provides resistance to one specific abnormal load has been challenged on a number of grounds. From an economic viewpoint, additional costs to prevent local failure under a

by complex methods, no practical analysis and design procedures are currently available.

It should not be construed from the above discussion that all abnormal loading conditions will always remain undefinable. Certainly loads imposed by earthquakes were not always specifiable. However, because of the high local risk, research was carried out and reasonable rational methods of determining design loads were developed and adopted within the various codes and standards. Similarly, tornado effects are currently being studied intensively(1), due to the increasing level of risk imposed by such loadings. It is anticipated that the semiprobabilistic approach currently being developed for tornado loading will, when completed, be adopted and enforced, especially for the more critical building occupancies. With regard to any new "normal" loading conditions, it is necessary that they be applicable to all forms of construction, and not arbitrarily limited to those which may "appear" to be more susceptible to their effects.

2.3.1.3 Alternate Path through Structural Integrity

Since the nature and magnitude of most abnormal loads are unpredictable, the third approach, of allowing local failure to occur but providing an alternate path within the structure to avoid a progressive collapse, appears to be a sound concept. However, as a consequence, a limiting (maximum) damage volume or area must be defined in some way.

When a structure has the ability to bridge over local failure

structure to mobilize reserve strength needed to resist progressive collapse.

The General Structural Integrity approach can be implemente in two distinct ways: given a damage volume or area the engineer can be required to apply a rational procedure (usi alternate path design) to establish the necessary integrity (direct design); or, alternatively, code writers can develo the necessary minimum detailing practice to establish a deg of continuity and ductility (indirect design). In either c it is necessary to identify the special sensitivity to abno loadings that the particular structural form possesses.(66-

Within this report minimum detailing is recommended as the most viable alternative.* In this approach, specific crite (in the form of tensile tie forces, deformability requireme wall layout, etc.) are given in the appropriate codes and specifications as minimum requirements for the design and construction of LP structures. Although this indirect appr is by its nature more general, it has a number of advantage

1. Code writers and researchers have a general respo sibility to evaluate details to assure the safety structures.

2. Design engineers should not be required to direct consider the effects of abnormal loads for one fo construction and not for another.

st recen ly his approach h s been don e by t C ittee on

minimum detailing requirements based on sound e
ing judgment can establish an adequate degree o
structural integrity.

Presently most building codes do not contain provisions n
to establish GSI in large panel concrete structures. Thi
shortcoming may be attributed to a combination of the fol
factors:

1. Building codes and standards have traditionally emphasized member design, while giving only lim guidance for the overall system design.

2. Design of connections has generally been omitte from earlier code specifications.

3. New forms of construction have been developed which connections are extremely critical.

While by its very nature LP construction does not lend it easily to a design with <u>moment</u> continuity, it is possibl develop <u>force</u> continuity and ductility of the connection well as of the overall system.(69,82,83,91-95) It is re that minimum requirements cannot possibly encompass all tions. Therefore, they should be supplemented with an e tional program to familiarize the engineering profession the problems encountered under abnormal conditions.

This Structural Integrity method, which has already been adopted in part by other countries, introduces a philoso design emphasizing strength and ductility of connections

The objective of the remainder of Section 2.3 is to present the concepts used in establishing minimum detailing criteri to provide continuity and ductility in the connections and structure. With the methodology established, experimental testing can then be performed to develop the necessary deta which will assure adequate force-deformation characteristics for the connections. It is anticipated that the continuity details will be specified as recommended practice, while the force-deformation characteristics will function as performa criteria when judging the acceptability of new materials and construction methods. Specific recommendations for both wi be given in subsequent reports.

2.3.2 Rationale for General Structural Integrity

Connections between the precast elements of LP structures are recognized as the weak link in this structural system. This weak ness is attributable to the lack of continuity through the connec tions, and to their brittle nature. Theoretically, friction-type connections can provide sufficient stability for gravity loads within an LP structure; however, such connections offer little resistance to the effects of abnormal loadings. Recent studies a discussions[69-74,78,97-101] on the performance of structures und normal and abnormal loads have reiterated that the integrity of a structure, regardless of the type of construction, depends on its connection characteristics; and that the strength of the elements cannot be utilized if the connections have insufficient strength. Therefore, as stated earlier, a degree of continuity across the connections and ductility within the connections should be ensure to achieve General Structural Integrity.

The magnitude of required continuity can be determined by assessing the forces acting within the connections under various conditions of local failure. Because of the nature of LP structures, these forces occur only as compression, tension, and shear.* Adequate compression and shear capacity in LP connections will usually be provided by design under normal loads. However, little tensile capacity is found between elements in LP structures, and therefore tensile capacity between elements across the connections (both horizontally and vertically) must be provided. This required tensile continuity across and within the connections can be achiev by providing the following ties (Fig. 16):

- transversal,
- longitudinal,
- vertical, and
- peripheral.

To develop a rationale to determine qualitatively the function of the ties and the required magnitude of tie forces, a realistic model must be selected to assess structural behavior following local failure. Failure mechanisms within this model should incorporate post-elastic material behavior with large localized deforma tions. The basis for minimum detailing must encompass the follow aspects:

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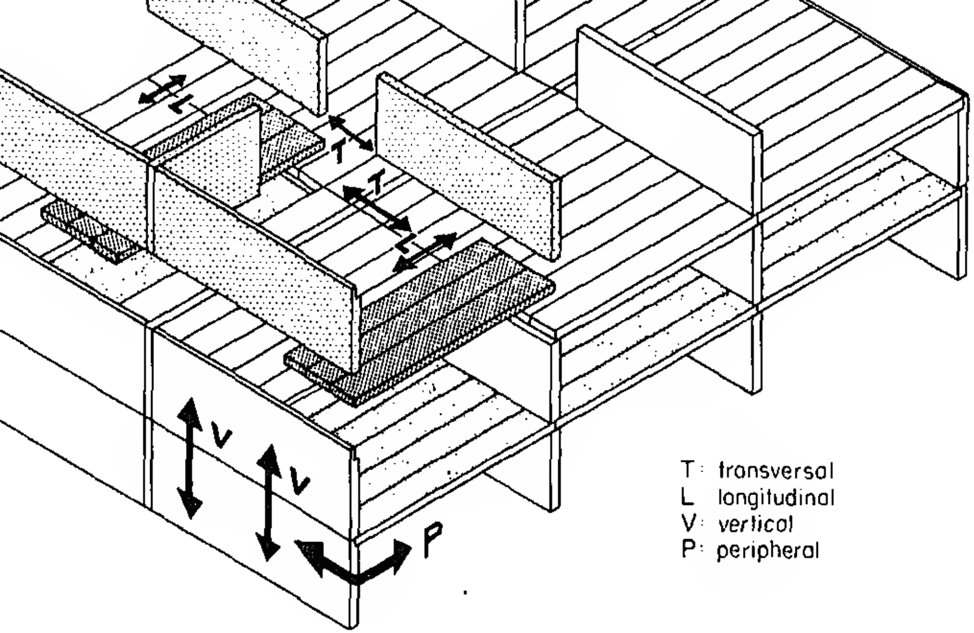
velopment of significant moment continuity between individual eleme not considered essential technically nor easily attainable economi the usual American-type LP systems.



16 Schematic Location of Tensile Ties in a Large Panel Structure

1. Extent of damage--an assessment of the nature and extent of local damage that is likely to occur under abnormal loadings;

2. Alternate paths--evaluation of alternate structural actions which can develop as a consequence of a local failure to re-establish load flow in the remaining undamaged structure--stability analysis of the partially damaged structure; and

3. Tie requirements--quantitative determination of requirements for transversal, longitudinal, vertical and peripheral ties to bridge local failures and to assure stability of the partially damaged structure.

2.3.3 Damage from Abnormal Loads

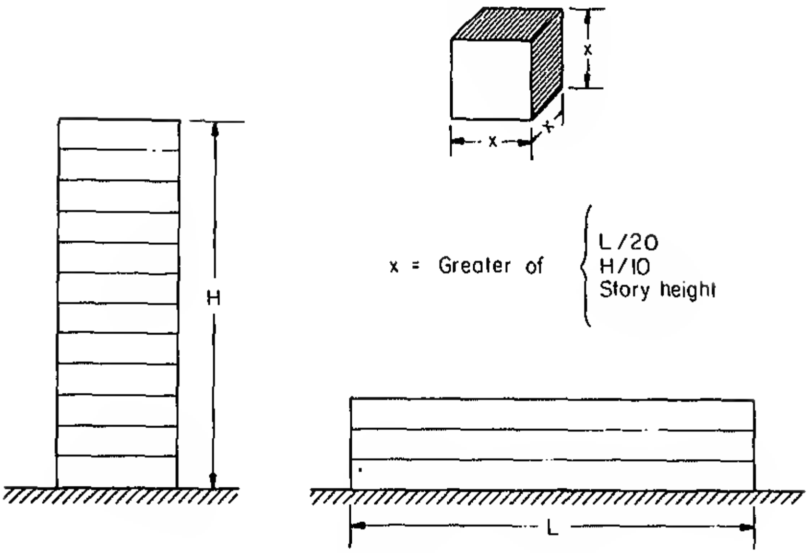
The indirect method of establishing General Structural Integrity in LP structures does not require the engineer to consider damage volumes (areas) or alternate paths in his design of the structure. However, to develop the minimum tie requirements to assure GSI, probable types and extents of damage must be defined. Reasonable definitions and limits of damage volume are suggested in this report only as a guide for a rational basis to determine tie requirements.

2.3.3.1 Damage Characteristics

Code writing bodies in the United Kingdom[59] and Sweden[96] have acknowledged that a certain amount of local structural damage is considered inevitable and acceptable by specifying the extent of local failure which the structure must be able to bridge (Fig. 17). The specified amount of damaged load-bearing wall is assumed "notionally removed" from the structure at any location within a given building.

The main premise when using the "notional removal" approach is that an element or portion of the structure has been totally removed. In LP buildings with brittle joints which depend primarily on friction and bond under compressive loadings, the "notional removal" concept may provide the basis for a realistic representation of their response to abnormal loadings. However, with ductility and continuity in the form of vertical

SWEDEN: DAMAGE VOLUME



NITED KINGDOM: NOTIONAL REMOVAL OF ANY VERTICAL COMPONENT

g. 17 Damage Criteria

With LP structures tied together horizontally and vertically, it is more reasonable to assume that wall panels become "ineffective" as a result of abnormal loading, i.e., they wil no longer function as load-bearing members as originally intended, but will remain in place in their damaged condition

Besides being considered more realistic, the concept of ineffective behavior is introduced because of its effect on debris loading and slab behavior in the damaged state. Specifically, the requirements for the slabs to "hang togethe

When estimating damage extent, the following must be taken into account:

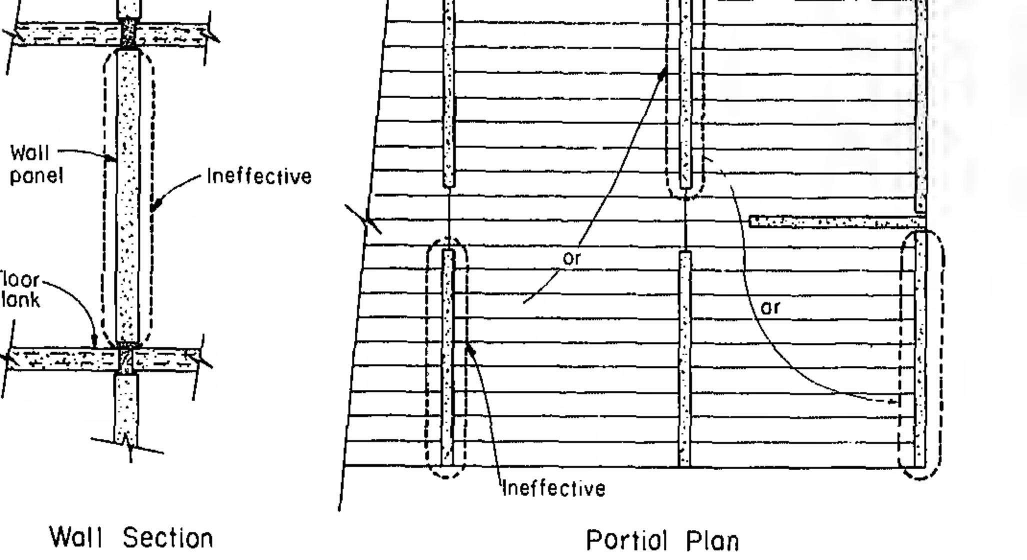
1. The consequence of failure of a particular element in the structure;

2. The influence of a particular structural configuration and layout of the walls; and

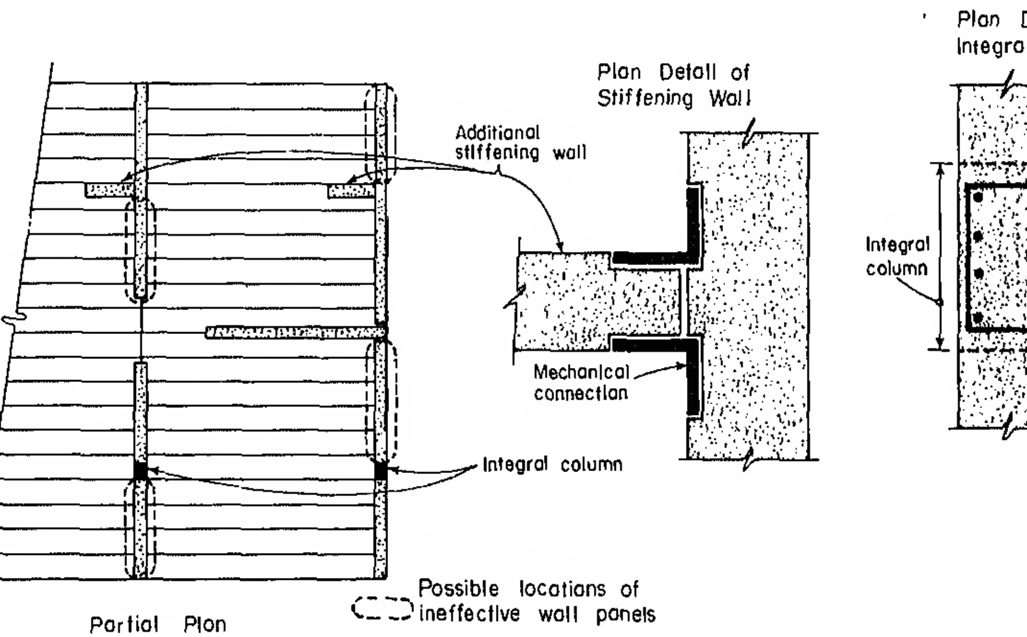
3. Load magnitude, if known or can be reasonably estima

Load bearing wall panels are the primary load transfer element in an LP structure. These panels which generally range in length from 25 to 45 ft., are "apartment" size rather than "room" size as is typical in European and Scandinavian constru tion. Because of the large size of wall elements used in American construction, it is unlikely that most abnormal loads would effect an <u>entire</u> wall panel(1). However, because walls are not generally braced at or between ends, it is reasonable to assume as an outside limit that an <u>entire</u> wall element located anywhere in the structure can become ineffective (Fig. 18). This somewhat stringent requirement is considered necessary due to the importance of the member in the structure

This requirement can be relaxed to a degree if measures are taken in the design to ensure that only a part of the wall becomes ineffective. Such measures include stiffening element in the form of wall segments or integral columns (Fig. 19). These stiffening elements however, must in turn be capable of



ig. 18 Extent of Assumed Maximum Wall Damage Under Abnormal Loads



psi (based on a gas service system explosion) is suggested as a minimum design loading for these elements.

2.3.3.3 Damage in Floor/Roof Panels

Floor/roof panels, or plank, are secondary load-transfer elements as they support only their own weight and superimposed loads. Their role with regard to overall stability, of bracing the walls against buckling, is less critical than the function of the primary load-bearing wall elements. Nevertheless, the damage to slab elements and the consequence of that damage must be assessed and controlled.

Slab systems typically consist of side-by-side members (planks) up to 8 ft. wide. As a result, full continuity of the system perpendicular to the span is interrupted at each joint. It is reasonable to assume, therefore, that in case of an abnormal loading, only a few floor/roof planks would become ineffective for a full span length. As the slab consists of individual planks, there is limited tendency for the failure to spread beyond the planks affected by the abnormal loading. For slab elements, "ineffective" suggests that they are physically damaged and substantially weakened, and no longer capable of providing lateral support of wall units or of participating in the required diaphragm action. However, they are still strung together and capable of supporting, at least in part, their own dead loads within their original spans.

A more critical condition for slab panels occurs when they lose their stability due to partial loss of one of their end supports, i.e., a wall panel. The loss of such support would

... slab spans for the slab elements to hang together their deflected shape. This catenary or suspension type action implies high ductility demands on the elements and t... at right angles to the bearing walls. Such action is neces... to reduce impactive debris loading on the story below.

2.3.3.4 Extent of Building Damage

Based on the above noted element ineffectiveness and assumi... that only one abnormal load occurs at a time, an estimated extent of damage which the structure should sustain without further collapse can be established.

If slab elements become ineffective, the damage extent is limited to those floor/roof elements which have been impair... If, however, a full size wall element becomes ineffective, ... damage will extend into the story above to include the slab elements supported by the affected wall. Thus the maximum damage volume which the structure is expected to sustain wou... extend vertically two stories (where the wall is ineffectiv... and the story above) and horizontally to the span on each s... of the ineffective wall.

2.3.4 Tie Functions and Alternate Paths

Structural design for "normal" loadings (those specified in code... begins with an assessment of the loads on the building. These loads should be safely transferred from the point of applicatio... the final resisting point through a logical load path. If a par... becomes ineffective as a consequence of an abnormal loading, a ... load path must be established in the remaining undamaged struct... i.e., an alternate path must be crea ed.

provisions should be made to effectively tie (or string) the slab elements together.

Tying the large panel structure together horizontally and vertically makes it possible to utilize the following structural mechan to bridge local failures:

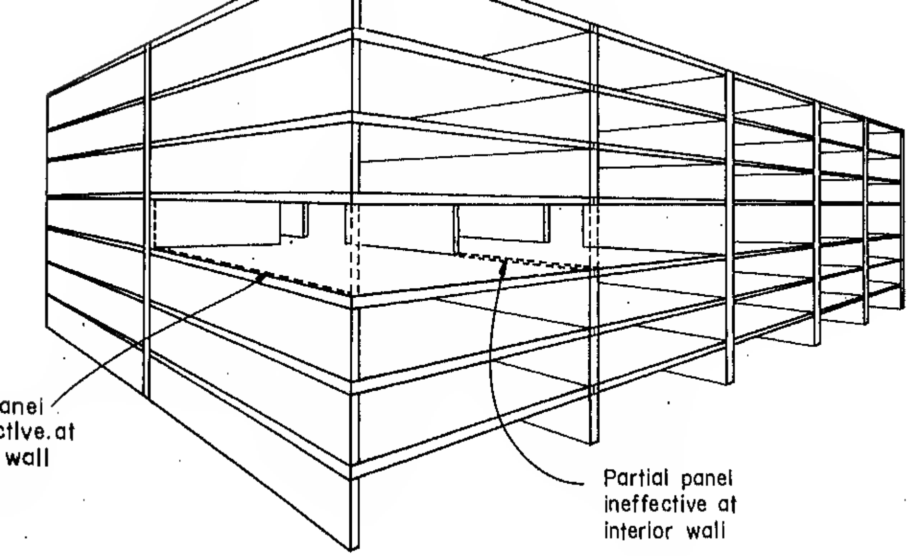
(a) cantilever action of wall panels;
(b) beam action of wall panels;
(c) partial membrane action of successive spans of floor planks;
(d) vertical suspension of wall panels; and
(e) diaphragm action of the floor planks.

2.3.4.1 Transverse Ties and Cantilever Action

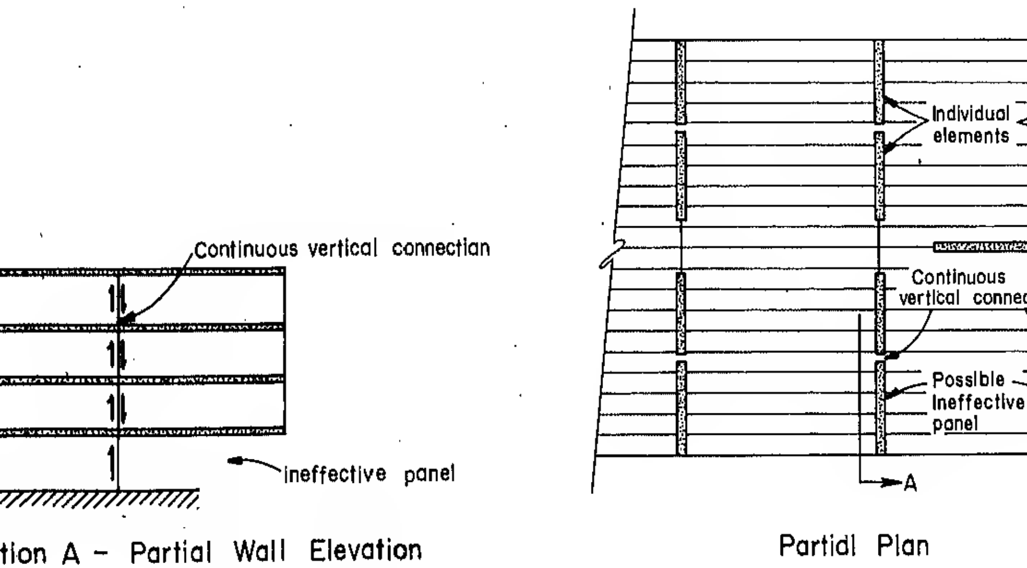
When, at a given floor level, a wall panel or portion thereof becomes ineffective (Fig. 20), the wall panels above that level lose their support. The most effective method of transferring the loads is through cantilever action of the remaini wall panels above. However, to obtain cantilever action, the vertical load must be supported at the cantilever root. This support can be provided by:

1. A vertical connection designed to carry the shear in each story in a wall assembly consisting of two or more adjacent vertical stacks (Fig. 21);

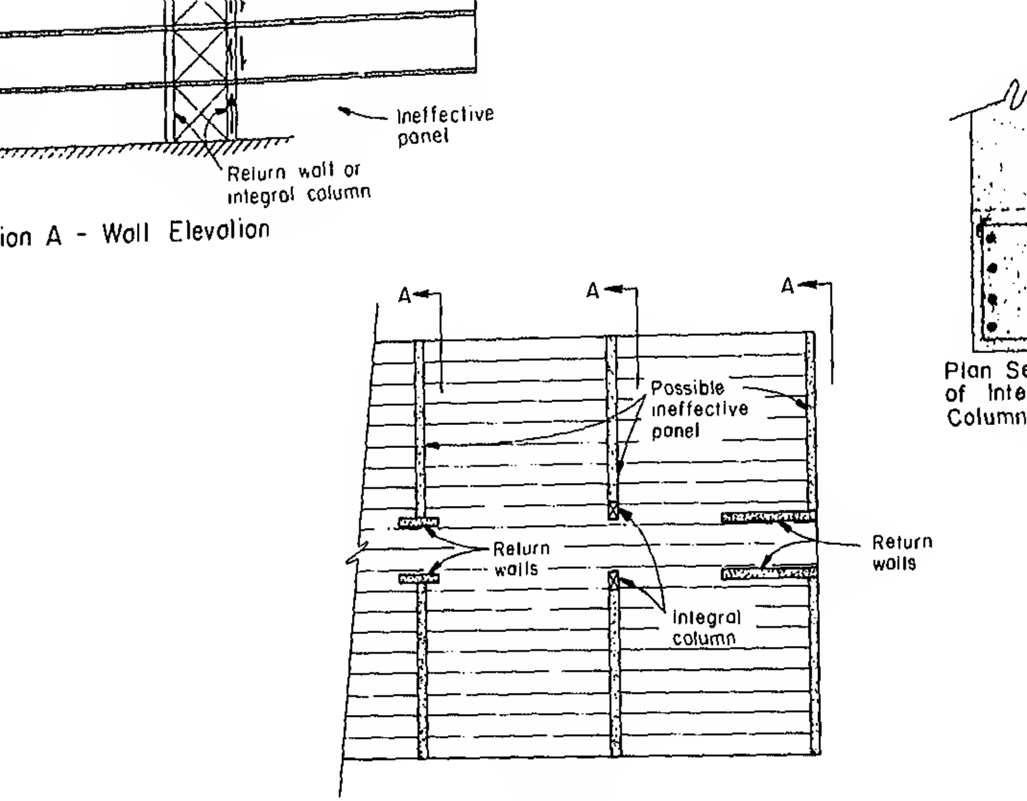
2. A vertically continuous return wall or a vertically continuous integral column at the interior edge of the wall assembly consisting of a single vertical stack (Fig. 22); or



20 Examples of Ineffective Panels

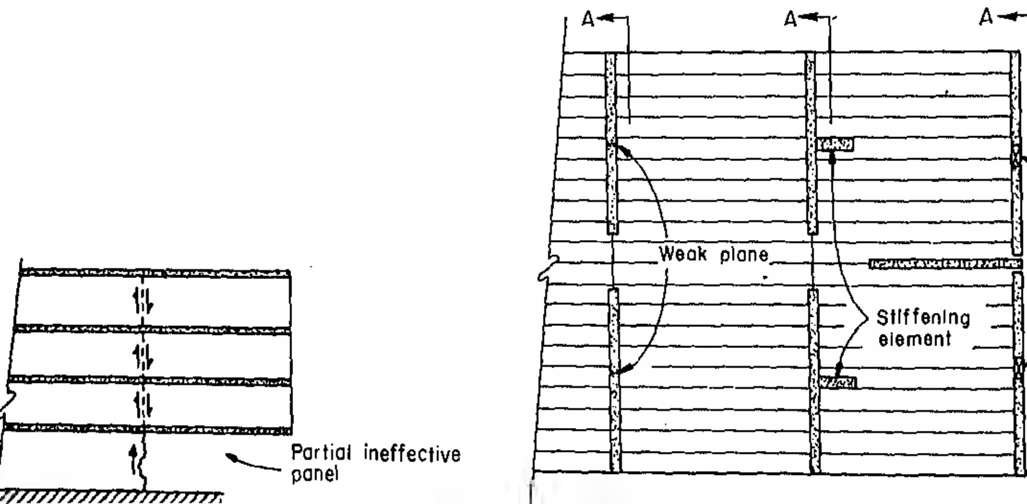


21 Cantilever Mode - Two Adjacent Wall Panel Stacks



Partial Plan

. 22 Cantilever Mode - Single Vertical Wall Panel Stacks



continue across all vertical joints between adjacent panel
In addition, adequate resistance against overturning of th
cantilever assembly must be ensured.

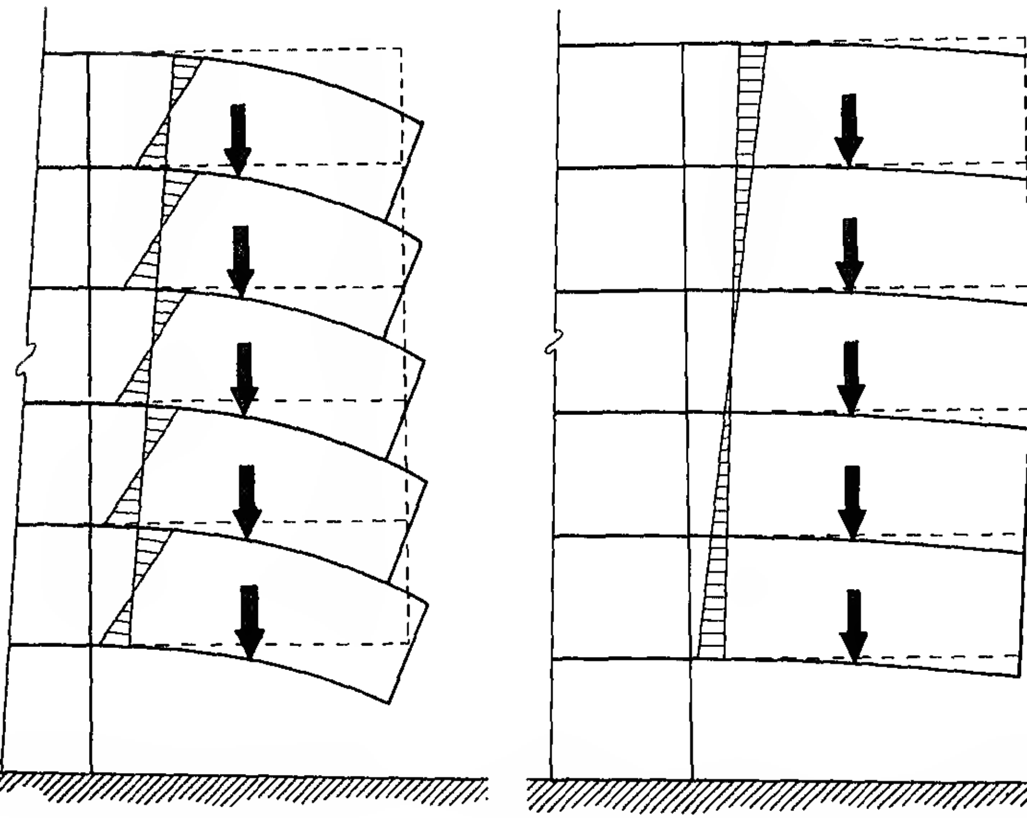
To assess the transverse tie force requirements and to dev
details to assure cantilever behavior, the following need
be considered:

(a) ability to transfer the tensile force of the hor
zontal tie at each story level into the wall pan
immediately below;
(b) shear characteristics of the interface between t
top of the wall and the bottom of the slab;
(c) shear characteristics of the interface between t
top of the slab and the bottom of the wall at th
horizontal joints between successive stories;
(d) shear characteristics of the vertical connection
providing cantilever support;
(e) shear capacity of the wall panels; and
(f) foundation-panel connection.

Of the above, shear characteristics of the horizontal inte
faces both above and below the slab, and the shear capacit
the vertical connections, affect the behavior of the canti
most significantly.

Depending on the horizontal connection properties, two mod
of cantilever action can be defined as giving upper and lo
bounds to the transverse tie force requirements. In the
cantilever action shown in Fig. 24a, no horizontal shear i
transmitted between story-high cantilevers. Thus, they ar
free to slide past each other on the interface between the

sive wall panels. ...
to provide shear resistance across the connections by shear friction, and consequently, enhance the cantilever strength. This behavioral mode provides a lower bound in predicting cantilever strength.



(a) No Shear Capacity in Horizontal Connections (b) Adequate Shear Capacity in Connections

Fig. 24 Modes of Cantilever Action

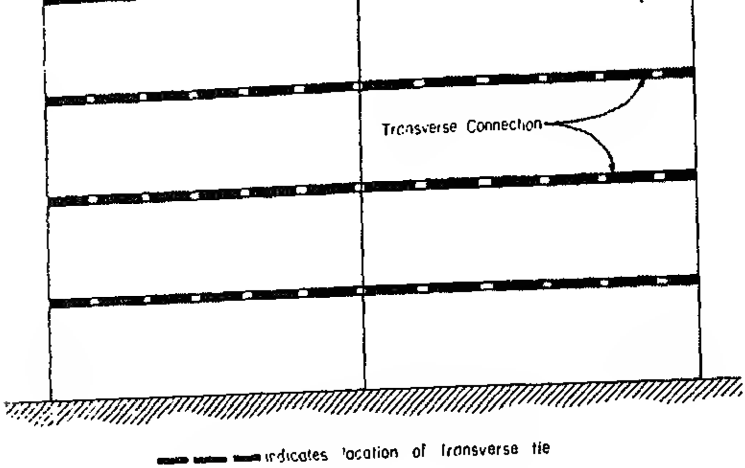
most likely occur within the vertical tie connection details. Therefore, this behavioral mode should be considered an upper bound in predicting the cantilever strength.

In practice the behavior will be somewhere between the two modes described above, and the tensile force requirements for the transverse tie in a multistory cantilever will depend primarily on the details affecting shear characteristics of the horizontal connections. Based on experimental tests to evaluate cantilever action, requirements for transversal ties and for performance and acceptance criteria for the horizontal connections will be given in a subsequent report on the analysis, design and acceptance criteria of connections.

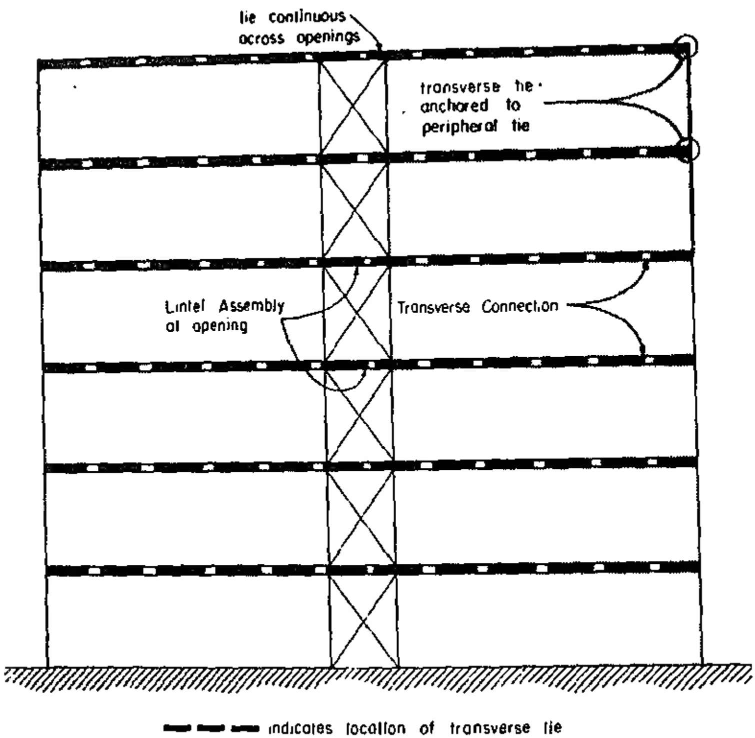
The transverse tie within the connection can be in the form of mild steel or unstressed prestressing strand. To ensure its effectiveness, the tie should extend the full width of the structure, and be adequately secured to the peripheral tie system (Fig. 25).

2.3.4.2 Transverse Ties and Beam Action

Although cantilever action through the use of transverse ties is the single most important element in establishing alternate paths in LP buildings, in some instances of local damage the transverse ties can also be very effective in developing "beam" action of the wall panels. However, to develop beam action, a capacity to transfer the vertical load to each of the supported ends must exist. Such supports are available in the cases shown in Fig. 26.



Flank Wall



Interior Wall

Fig 25 Suggested Extent and Location of Transverse Ties

ineffective panel

Section A - Wall Elevation

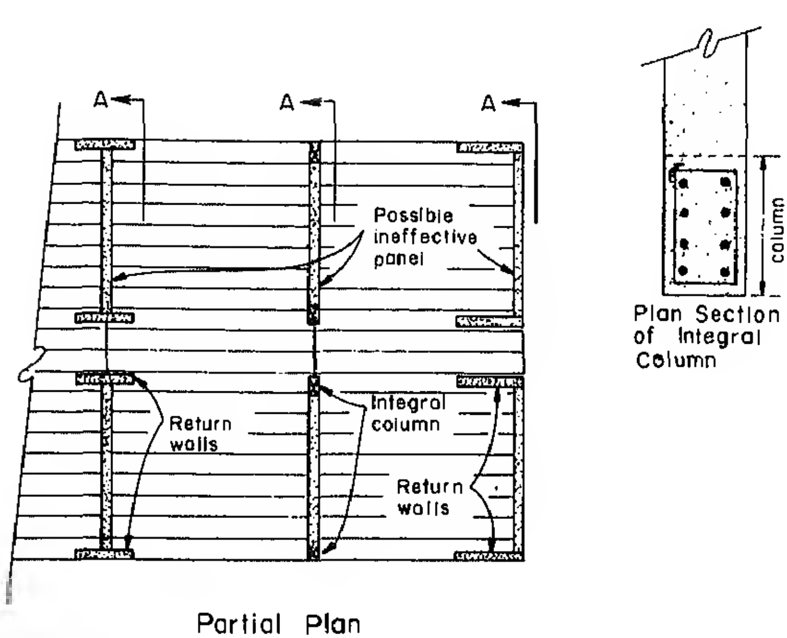
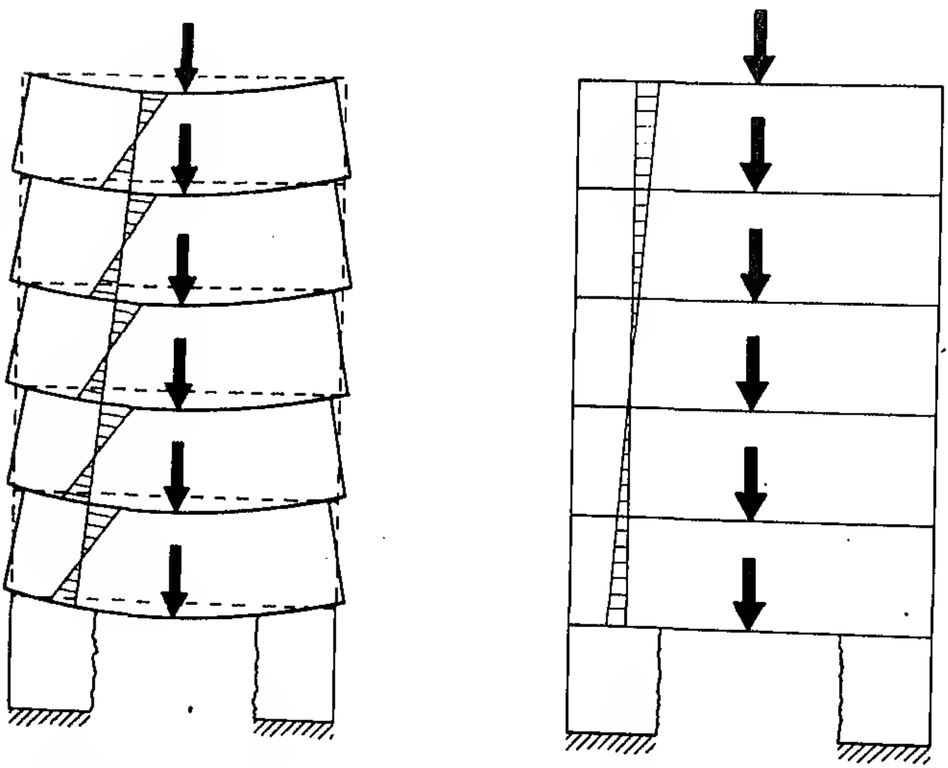


Fig. 26 Beam Mode - Vertical Wall Panel Stacks

As in the case of cantilever action, the vertical su
beam action are provided through proper panel design
layout. The beam's flexural resistance can be devel
if adequate tensile capacity in the form of reinforc
exists in the transverse horizontal connection or wi
wall panels themselves. Again this continuity would
across all vertical joints between adjacent panels.
factors affecting the overall behavior of this alter
flow are similar to those examined under cantilever
Vertical load-carrying capacity at each end must be

27a, the horizontal connections offer no resistance against relative slip between the consecutive story-high wall panels. Thus, the wall panels slide past each other, and the total strength of the beam is the sum of the flexural strengths of the individual wall panels. Under these conditions, rather than transverse ties, tensile reinforcement can be embedded in the panel itself. This differs from the case of cantilever action where the tensile capacity must be provided by the transversal ties embedded in the horizontal connections.



(a) No Shear Capacity in Horizontal Connection

(b) Adequate Shear Capacity in Horizontal Connection

Fig. 27 Modes of Beam Action

tions offer full resistance against slippage and consequently the beam acts as a monolithic element. This behavioral mode is an upper bound of the beam strength.

The necessary ties to resist the tensile forces from the bending moment will be less than those required under similar span cantilever action; however, this structural behavior will usually require more return walls or integral columns to assure vertical supports. The transverse ties to effect beam action can be either of mild steel reinforcing or unstressed prestressing strand. The ties should be continuous the full width of the building and anchored to the peripheral tie system (Fig. 25).

2.3.4.3 Longitudinal Ties and Partial Membrane Action (Large Deflection of Slabs)

Catenary action of the continuous slab spans due to loss of supports has been investigated in Europe.[82,94,102-104] Although full catenary action is accepted as an appropriate and functional means of re-establishing the load flow in European LP structures, it is not considered in this study a suitable method for American LP construction for the following reasons:

1. The typical slabs for European systems span from 10 to 18 ft., are cast for a particular building, and are joined together by interlacing loops protruding from the slabs. LP systems used in the United States have floor spans which range from 20 to 40 plus feet. The U.S. slab systems are generally

necessary ... slab span) is about twice that of its European counterpart, the problems related to physical development of the necessary longitudinal tensi forces are greatly magnified.

2. To develop the necessary catenary action at rea force levels, the floor systems must undergo la deflections. With 40 to 80 foot catenary spans U.S. construction, the necessary deflection ap a total story height. However, to develop alt paths through story high deflections of slabs simply not acceptable due to serviceability co ations for the structure in the damaged state.

The major element of resistance and stability in the dar wall-type structure is provided by cantilever and beam a Therefore, should an interior wall become ineffective, only function of the slab is to ensure a stringing toge its elements to inhibit progressive collapse from debri loading. This is necessary since the slabs are general not tied directly into the wall above.

To string the slab elements together in a deflected sha (Fig. 28), tensile continuity must exist between them, the ineffective wall. Ductility of the connection must be insured to satisfy the large deformation demand and resist the dynamic effects that may occur. In a catena structure of this type the deformation (or ductility) c increases with increased deflection while the tie force decreases.

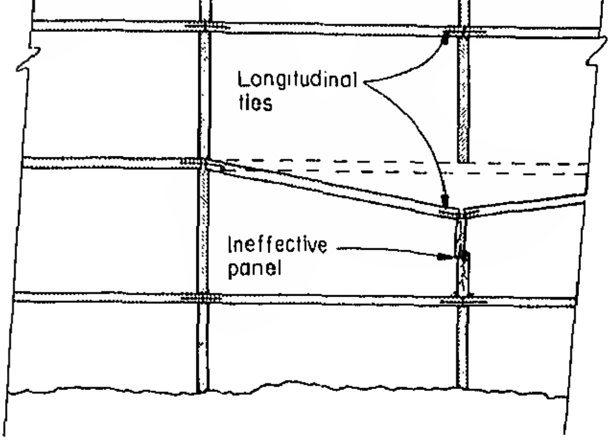


Fig. 28 Partial Membrane Action of Floor Elements at Ineffect
Panel

Continuity can be developed with reinforcing placed in the longitudinal horizontal keyways between floor panels, or in the topping slab if one is employed. To ensure integral behavior, the ties of the end span should be anchored into th peripheral tie system. Since floor elements are well reinforced, the ties need not be continuous for the full length; the tie force should be transferred by bond or mechanically from an element in one span to an element in the next span (Fig. 29).

The elongation in the plane of the slab required to achieve the large deflection at the ineffective support must take place primarily in the ties at the supports. Parameters affecting the elongation characteristics and the overall effectiveness of the tie within the joint include:

(a) keyway shape and size affecting its longitudinal shear capacity;

parameters on the overall strength and ductility of the lo
tudinal ties will be evaluated. The tests have a two-fol
objective: (a) to form the basis for establishing minimum
force requirements and performance criteria for various
key joint configurations presently in use, and (b) to dev
acceptance criteria for newly developed details. The tie
requirements and criteria will be examined in detail in a
subsequent report.

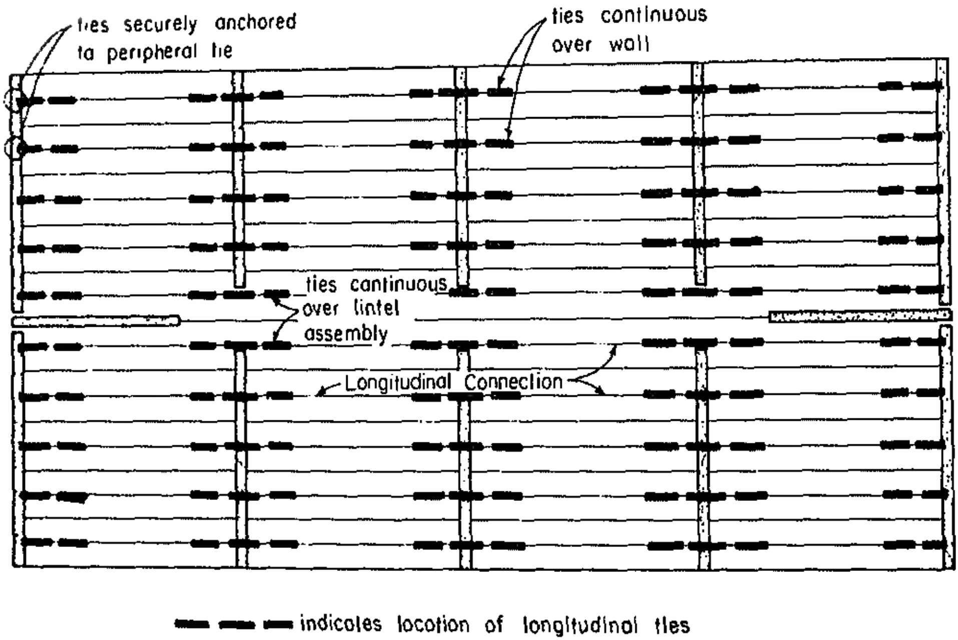


Fig. 29 Suggested Extent and Location of Longitudinal Ties in Flo
Roof Panel Assembly

1. Provide vertical suspension of ineffective wall panels to limit debris loading.

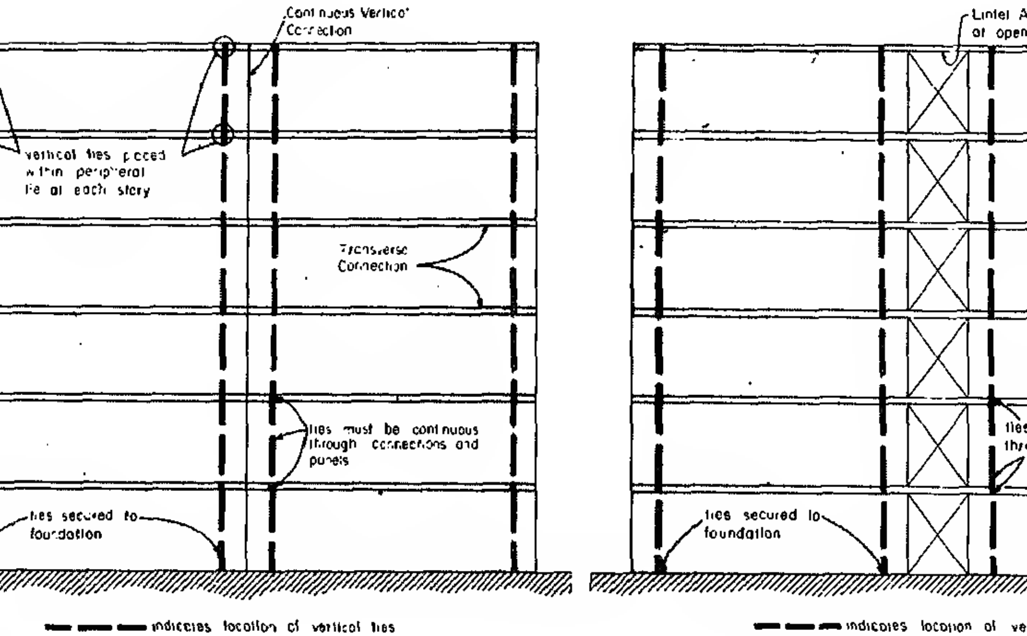
2. Provide resistance against "kicking out" of the walls sideways, thus fostering "ineffective" behavior of wall elements rather than total removal under abnormal loading.

3. Assure clamping and dowel action in the horizontal connection for shear friction to resist horizontal shear and thus develop cantilever and beam action.

If an alternate path for the load transfer around an ineffective wall panel has been established either through cantilever or beam action, the damaged panel should be suspended from the alternate supporting mechanism. To ensure this suspending action, vertical continuity in the form of tensile ties must exist between and through all wall panels. Tensile continuity from foundation to the roof can be assured by providing vertical ties proportioned to resist (within each story) the dead loads of the wall plus those loads superimposed by the floor panels. In addition, the vertical tie must resist the stresses from shear friction clamping in the horizontal connection, as discussed in Sections 2.3.4.1 and 2.3.4.2.

Because of the increased vulnerability of exterior wall panels (as compared to interior wall panels) with regard to abnormal loadings, greater vertical continuity is desirable in flank wall assemblies in order to afford them increased protection against removal from the structure. The vertical tie of the flank walls should be placed within the peripheral

(Fig. 30). The ties can also function as reinforcement required for service and erection loads.



Wall Interior Wall

0 Suggested Extent and Location of Vertical Ties

2.3.4.5 Peripheral Ties and Diaphragm Action

Peripheral ties in the damaged building should:

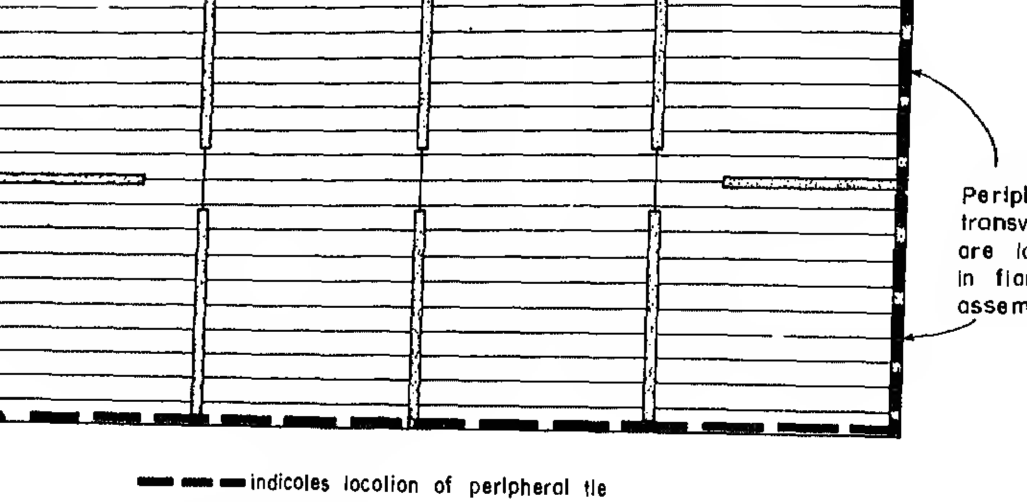
1. Establish the necessary diaphragm action to resi the effects of wind, torsion and unequal load distribution throughout the structure;

These functions can be accomplished by employing a ring beam at the periphery of the structure at each floor and roof level.

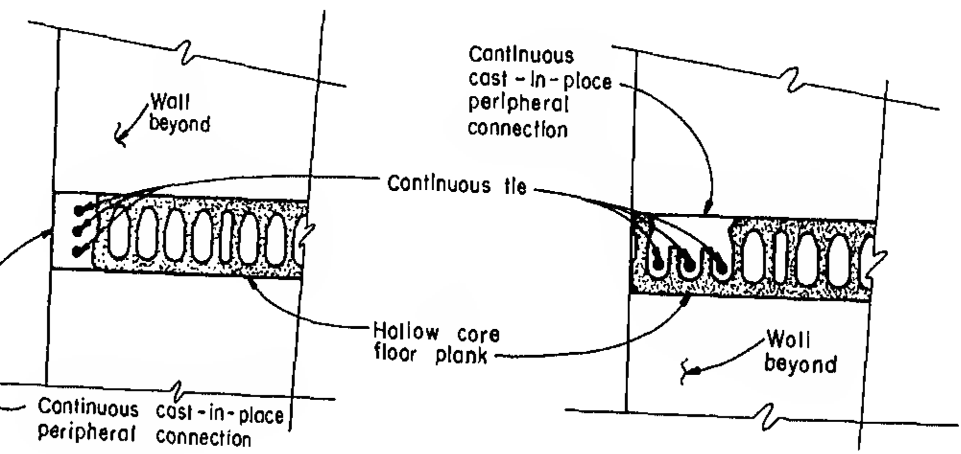
To assess the force level required in such a ring beam in a structure is a difficult, if not impossible, task. In the damaged state the entire slab or roof may act as a membrane, with its periphery subject to large ring beam forces. It is suggested that the peripheral tie forces be based on the diaphragm action necessary to resist normal lateral loads, assuming the customary load factors and capacity reduction factors. However, for purposes of continuity and anchorage requirements in the damaged state, the transversal tie forces required in the flank wall assemblies should be continued in the longitudinal faces (if larger than those required for diaphragm action) for the design of the complete periphery (Fig. 31). These continuous ties, which should be located as close to the actual periphery as possible, may be of mild steel or prestressing strand (Fig. 32a,b), or mechanical connectors between ends of planks in adjacent spans (Fig. 32c,d).

2.3.5 Debris Loading

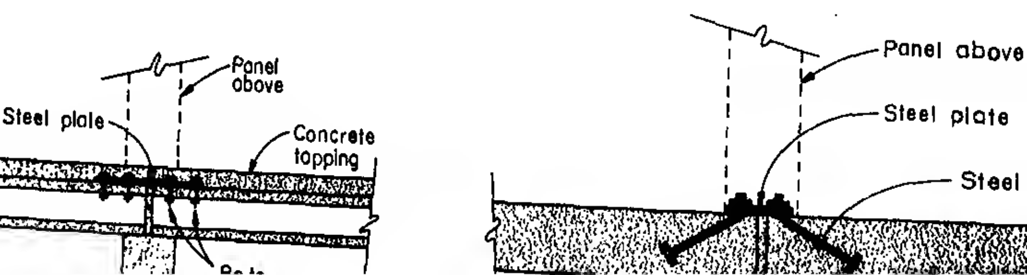
Although ties are provided to hold the elements together, when a floor/roof panel is subjected to an abnormal load, the possibility of impact loading from spalled fragments still remains (Fig. 33). The magnitude and nature of such debris loading depends upon the mass and velocity of the fragments, as well as their shape. These variables in turn are functions of the modes of collapse of the

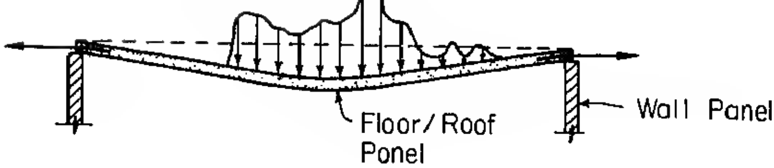


31 Suggested Peripheral Tie in Typical Floor/Roof Panel Assembly

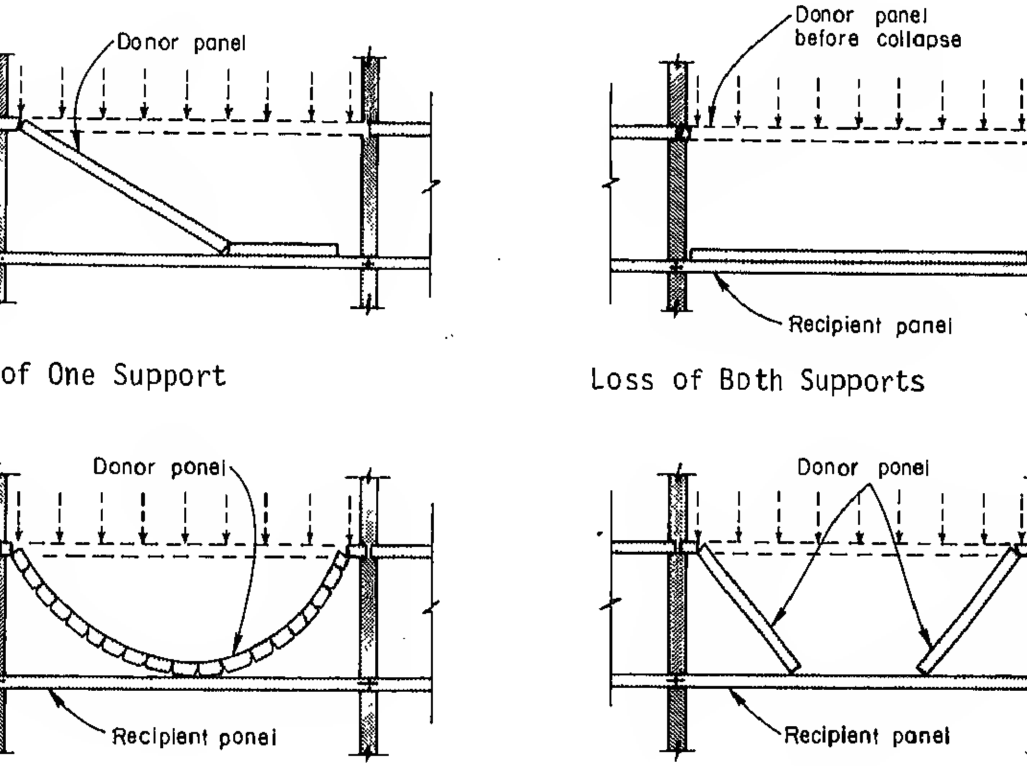


(a) (b)





ig. 33 Floor/Roof Panel Subject to Debris Loading



of One Support

Loss of Both Supports

Deflection at Midspan

Fracture at Midspan

34 Modes of Floor/Roof Panel Collapse

For convenience, the slab panel which causes debris loading has
termed a "donor" panel, and the panel which is subject to the de
loading has been termed a "recipient" panel.

earlier. Further, the connection details developed ... which will enhance the ductility for all connections, will b... specified as recommended practice. As a result, except for ... situations, it seems unnecessary to consider debris loading ... in the design process.

2.3.6 Loads and Safety Factors in the Damaged Structure

To determine the quantity of ties needed for stability of th... damaged structure, the necessary loads to be resisted must b... established. Consideration should be given only to those lo... likely to occur before temporary or permanent measures are ... repair the damaged area. Included in this category are the ... dead load and part of the design wind and live loads.

Load-frequency distribution curves for both live and wind l... time dependent. As a result, reduced live and wind loading ... suggested, based on the supposition that the building will ... the unsupported damaged state for a limited period of time. ... values ranging from 10 to 50% could be recommended dependin... the assumptions made. In the absence of a definitive basis... value of 1/3 the specified live and wind load is arbitraril...

Since the rate of occurrence of local damage due to abnorma... ings is relatively low, and since the intent is to assure t... stability of partially damaged structures, the above loads ... resisted at a level close to the ultimate strength of the s... say 95% of ultimate capacity. Therefore, for the determina... minimum tie forces, liberal load factors, γ, are appropria... However, the capacity reduction factors, φ, which account ...

Combining the loads (D, L, W) and load factor, γ, the required strength equation may be expressed as:

$$U = 1.05\ (D + L/3 + W/3) \qquad \text{(Eq. 2.3}$$

Incorporating the capacity reduction factor, φ, the safety fact of the damaged structure becomes:

$$S.F. = \frac{\gamma}{\varphi} = \frac{1.05}{0.9} = 1.17 \qquad \text{(Eq. 2.4}$$

2.3.7 Specifying Tie Forces - Unstressed Strand

Specifying tie requirements in terms of forces rather than amou of steel permits flexibility in the choice of tie type, while fostering a better understanding of the technical issues involv Specifying forces allows the use of unstressed prestressing str which has the advantage of large tensile capacity with a relati small diameter. Flexible strand also offers the advantage of e placement in misaligned joints where a normal mild steel bar cannot be placed due to its rigidity. While unstressed prestre ing strand cannot be used for resistance at service load levels since it would result in unacceptable deflections and crack wid the large strains of the strand are not objectionable under cat trophic conditions.

2.3.8 Plan Layout

Proper structural planform, i.e., the layout of the structural walls, improves the overall rigidity and stability of the struc thereby increasing its structural integrity.[105] Effective structural layout is best accom lished b arranging walls in

tion. They will also improve the stability of individual transve sal walls, while decreasing the length of wall likely to be affec by an abnormal load.

For exterior walls, returns or integral columns are desirable to improve their resistance to abnormal loadings. If vertical stab lizing cores are the only elements for lateral stability, it is desirable to distribute them throughout the building (Fig. 35) so that if one is damaged, total stability of the structure under lateral loads is not forfeited. Shrinkage, temperature and distr bution steel placed in the topping can also be used to enable the slab to span in a perpendicular direction and prevent collapse where a support configuration is favorable for two-way action (F 36).

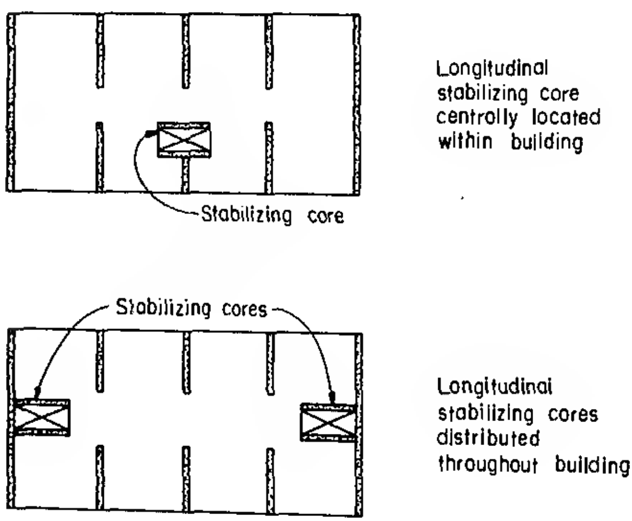


Fig. 35 Possible Core Layouts in Typical Cross Wall Structure

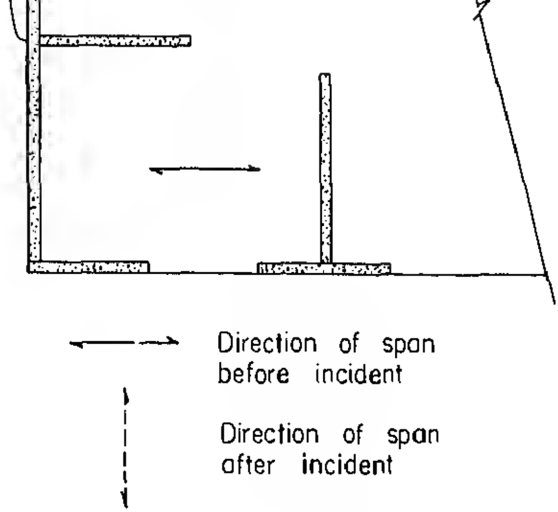


Fig. 36 Change of Slab Span Direction Under the Effects of Abn
Loadings

2.3.9 Concluding Remarks

It is technically very difficult and economically prohibitive t design residential-type buildings for absolute safety. On the other hand, there is no justification for constructing building which do not afford a degree of safety with regard to abnormal loads.

To reduce the risk of progressive collapse of large panel resid tial buildings, a philosophy is developed to assure bridging of local damage while maintaining stability of the partially damag structure by tying the components of large panel concrete struc together horizontally and vertically. Based on this philosophy explicit requirements for a minimum tie system of transversal, longitudinal, vertical and peripheral ties will be developed.

to design for any particular abnormal load, since the ability to
bridge local damage is provided.

In establishing structural integrity, the need for tensile conti-
nuity and ductility of the elements and their connections, as
as of the overall structure, is recognized. This will be acco-
plished through a rational arrangement of tensile ties (Figs.
37,38) and through connection details which will assure an alt-
nate structural path in the damaged building. This combinatio
system continuity and ductility should enable the structure to
either absorb the abnormal loads with minimal damage, or bridg
localized damage as a result of the abnormal load. The provis

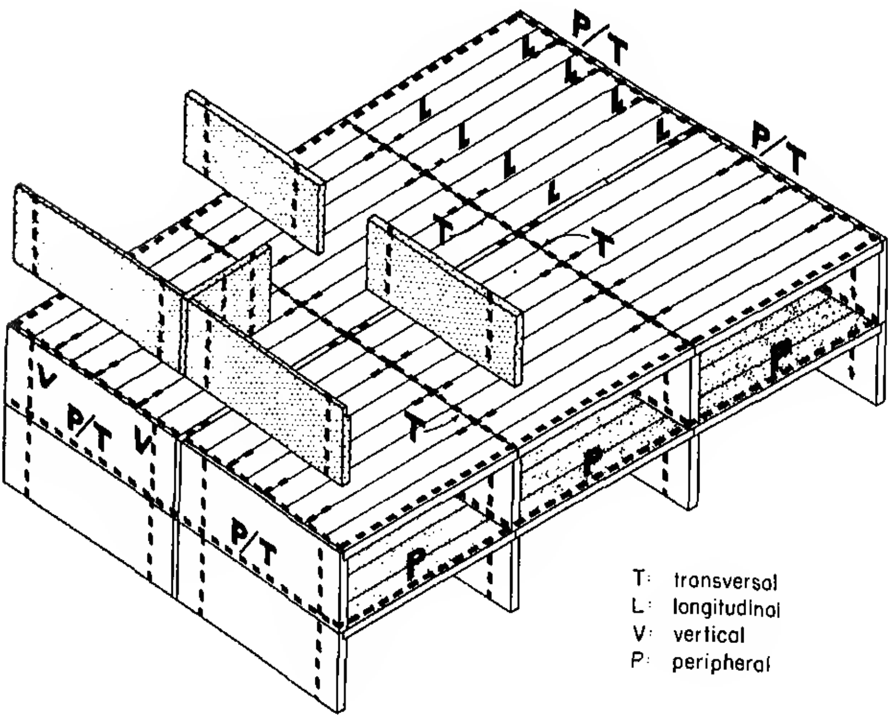


Fig. 37 Suggested System of Tensile Ties in Large Panel Structure

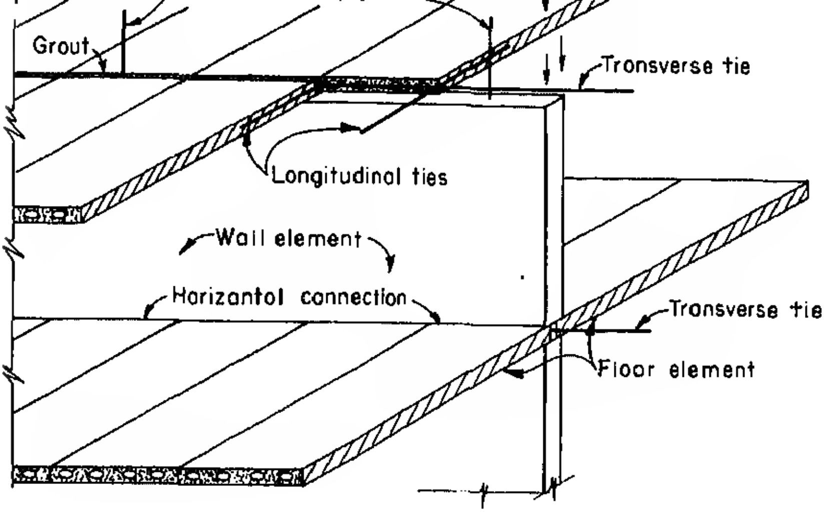


Fig. 38 Ties at Interior Wall-to-Floor Connection

of General Structural Integrity will bring the safety of LP st tures closer to that of the traditional cast-in-place reinforc concrete buildings.

The objective of this approach is to limit and substantially reduce the general risk of collapse, as compared to that exist if no such measures were taken. It is not intended to afford absolute safety in regard to any exceptional event in any part every building. Based on experimental tests performed thus fa appears that the amount of ties required to achieve the desire level of General Structural Integrity will have a minimal effe the economics of LP structures. With the philosophy establish subsequent experimental tests and analytical studies will focu development of purposeful details to optimize the effectivenes the ties in the structure.

late the allowable deviation or "tolerance" from a specified dimensio[n]. A reasonable system of tolerances is an economic and technical necessi[ty] for the success of any precast system; however, despite their vital importance, tolerances have frequently been neglected.[52,56,106-109]

Tolerances are determined by the requirements of the entire constructi[on] process considering technology and economics of currently used methods of precasting, setting-out and assembly. The most influential factor affecting tolerances appears to be the connection type--more accuracy [is] required for bolted connections than for grouted connections. Tolerances should guarantee correct assembly and efficient functioning of individual precast panels. Each panel is positioned within the basic space allotted it and should not encroach on the space allotted to another panel. Therefore, it is only necessary to fabricate and erect LP structures accurately enough to assure that the deviations do not fall outside known and acceptable limits.[107,108,110]

Little information is available on the cost of achieving various degre[es] of accuracy, though it is generally accepted that cost increases with accuracy. Most recommended practices[26,53,111-115] establish the permissible dimensional deviations in individual precast panels and in th[e] panel assembly, and leave the methods for achieving accuracy to the discretion of the fabricator and erector. No recommendations as to ho[w] tolerances should be accounted for in design are available.

Requirements for dimensional accuracy in the design of connections, f[or] for assembly of panels, and architectural details have contributed to excessively small specified tolerances. As a result some current standards have been criticized as illusory, stringent, and vague,[56, and consequently, independent tolerance systems are in use.[56]

terial deviations are caused by ambient temperature, moisture o
resses in the panels. Such deviations can occur during or afte
nufacture of the panel and should be taken into account when me
panel for conformity with the plans. These deviations are gene
all in magnitude and their computations straightforward.

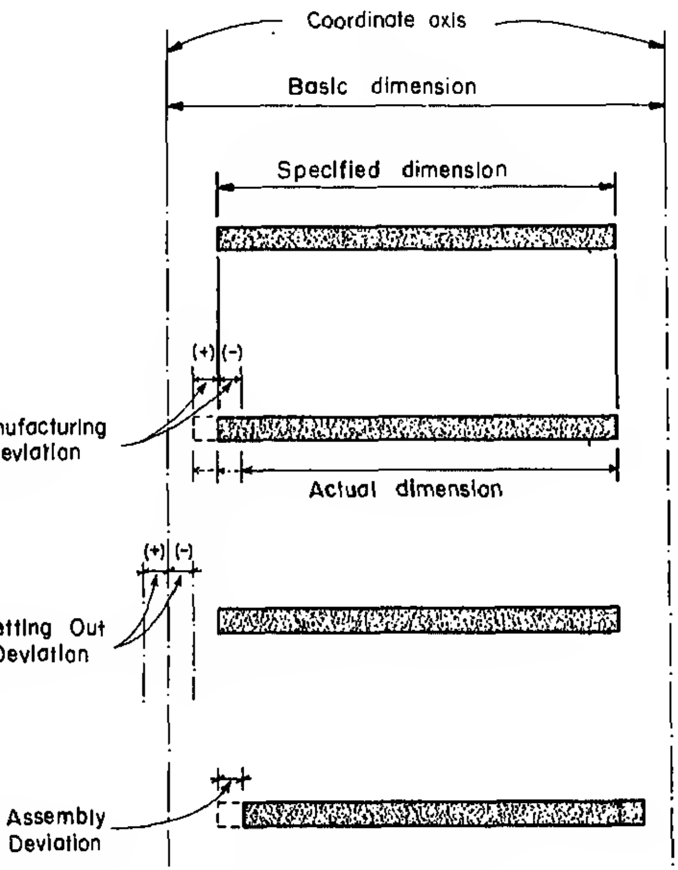


Fig. 39 Panel Size Deviations

n practice these deviations or variations are caused by human error as
ell as the limits imposed by measuring instruments and techniques.

2.4.1 Manufacturing Deviation

Three types of dimensions of precast panels are shown in Fig. 39, namely:

(a) basic or coordinating,
(b) working or specified, and
(c) actual dimensions.

Basic dimension denotes the dimension between the axes defined by the two-dimensional grid (a two-dimensional coordinate system of reference lines defining the layout of the LP structure). The working dimension is the planned dimension of the panel arrived a from both its basic dimension and joint dimensions. Actual dimen sion refers to the measured panel dimension after production. It differs from the working dimension due to manufacturing and mater deviations.

Manufacturing deviations are caused by production method, equipment, formwork (mold), workmanship, and quality control. At present wall panels are cast vertically or horizontally in molds while hollow-core floor plank is either extruded or cast by speci processes. The deviations which occur can be in each of the panel's three coordinate axes. Additional deviations that may affect elements include angularity, bowing (concave or convex), a warping (Fig. 40).

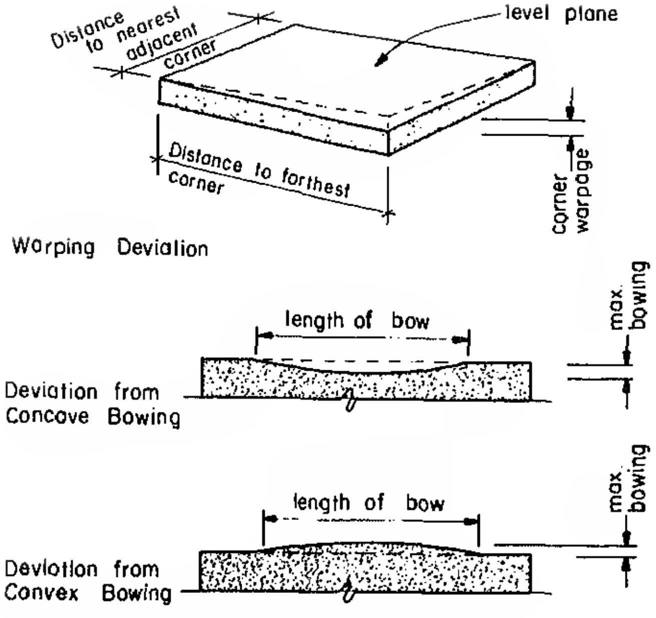


Fig. 40 Manufacturing Deviations

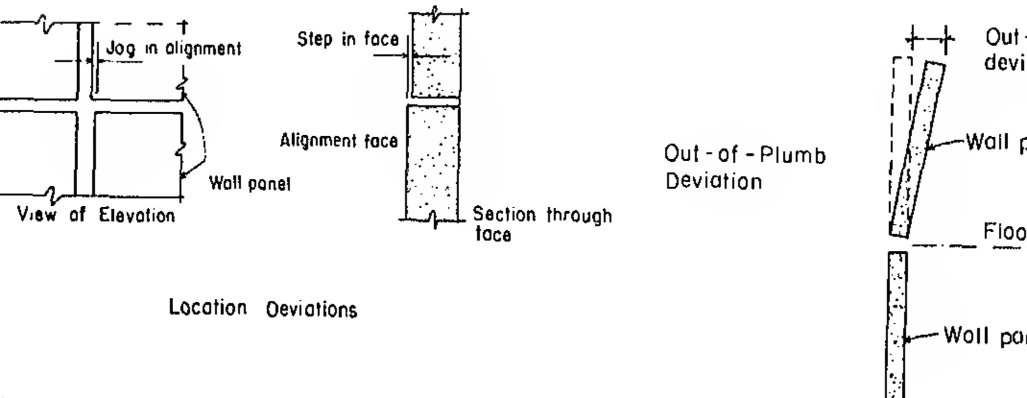
Errors in mold dimensions at the time of casting, errors due
incorrect filling of a mold, and the changes in the dimensio
the panel after compaction are considered to be the main sou
fabrication deviations for wall panels.(108,116-118) In flo
elements, length error, caused by inaccuracies in cutting lo
panels into short pieces is the main manufacturing deviation

2.4.2 Setting Out Deviation

The measured space into which a panel is designed to fit is
to setting-out deviations as shown in Fig. 39. These deviat
are generally the result of limitations of measurement metho
equipment. The importance of setting-out deviations has bee
recognized and some guidance provided.(113,119-121)

out of line, it may be rotated in plan, or it may be out-of-plumb (Figs. 39,41). As panels are placed one after the other, their dimensional deviations can compound or cancel depending upon the direction of deviation. Assembly deviation can be predicted by u of "statistical theory of accumulated errors".(108,110,117,118,12

At present little information is available regarding the factors affecting assembly deviations, however, the weight and maneuverability of the panels seem to have some effect.



ig. 41 Positional Deviations

2.4.4 Table of Tolerances

A statistical solution for tolerances is not yet available. However, as a result of extensive experience, tolerances are becoming standardized in various countries through recommended practice and codes.[26,52,108,112,113] Based on a survey of several tolerance systems used in North America, the following tolerances seem to be reasonable:

Position of voids	± 1/4 in.
Position of tendons	± 1/8 in.
Position of weld plates	± 1 in.
Camber deviation from design camber after jacking	± 1/8 in. per 10 ft. not greater than ± 3
Differential camber between members of the same design after setting and alignment	1/4 in. per 10 ft. b greater than 1/2 in.
Squareness of ends - vertical and horizontal alignment	± 1/4 in.

Dimensional Tolerances for Wall Panels

Length	± 1/2 in. up to 40 f
Height	± 1/4 in. up to 10 f
Thickness	± 1/8 in.
Sweep or Bowing	3/8 in. up to 40 f
Opening (Location/Size)	± 1/2 in.
Position of handling devices	± 6 in.
Position of weld plates or connection devices	± 1/2 in.
Diagonals	± 1/2 in. from plan

Location Tolerances for Precast Panel Assembly

Face width of joints	± 1/2 in.
Joint taper	1/40 in. per ft. le 1/4 in. maximum len ing in one directio
Step in face (Fig. 41)	1/4 in.
Jog in alignment of edge	/4 in.

Due to production techniques, walls are generally more expensive than slabs on a per square foot basis. Hence, care is usually taken in layout and analysis to optimize the wall-to-slab ratio LP structures.

Typical wall elements in LP structures range in thickness from 6 12 in., in length from 10 to 45 ft., and in height from 8 to 10 In general, wall panels have a section characteristic (i.e., sol ribbed, cored, etc.), and a function characteristic (i.e., trans verse or spine, interior or flank walls). The principal require ments for a load-bearing wall panel are adequate strength and deformability to satisfactorily accommodate both normal and abno loads. In design of wall panels the following considerations should be included:

- load and geometric eccentricities;
- slenderness effects;
- effects of connections;
- effects of openings;
- thermal effects; and
- minimum thickness.

A detailed study of wall elements is presented in Report 3: "Wal Panels: Analysis and Design Criteria."

2.5.2 Effects of Clamping on Simply Supported Plank

Floor elements in LP residential structures in North America are mostly hollow core, precast, prestressed plank up to 8 ft. in width, of the cross-sections shown in Fig. 42. Structurally, f

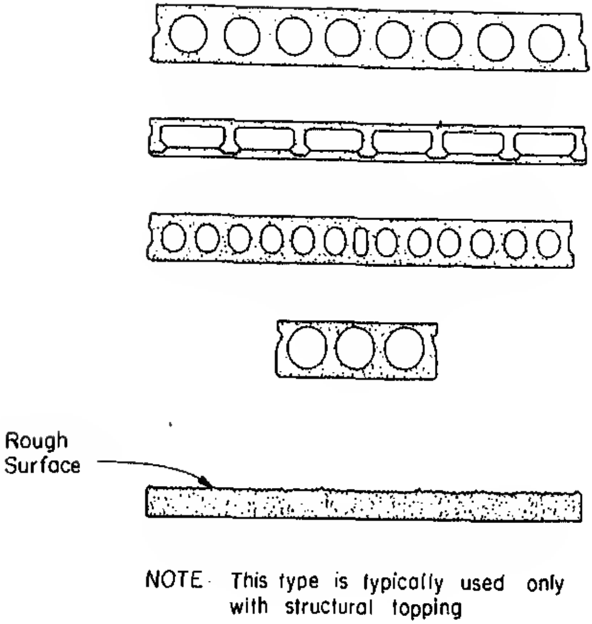
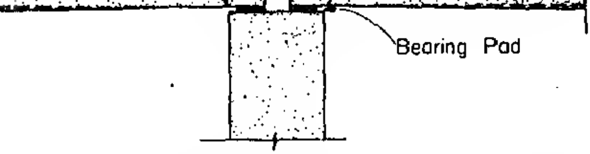


Fig. 42 Typical Cross Sections of Floor Plank

Floor planks are usually designed as simply-supported one-way slabs. The panels placed on the walls are free to deflect due to their self-weight. After the space between the top of the floor panels and the underside of the wall panel is drypacked, the end rotation of the floor panel is restricted by clamping action. Thi induces a negative bending moment over the support for loads subsequently applied and has the beneficial effect of reducing midspa deflection. However, when the bending moment exceeds plank sectional capacity, cracks may develop in the top of the plank near the wall faces (Fig. 43).



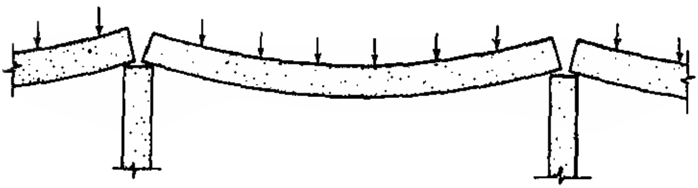
g. 43 Cracked Floor Panels at Supports

ecent unpublished tests[(122)] of slab panels on elastomeric bearing ads indicate that for superimposed loads typical of residential cupancies, the cracks (if any) at the supports are narrow and do ot penetrate to the reinforcement level. No perceptible reduction shear capacity of the plank was found. However, the results so suggest that where the ratio of superimposed load to dead load ubstantially exceeds one, the shear capacity is substantially educed as the ultimate moment capacity is approached. In such ases to avoid dangerous shear failures, reinforcement should be rovided at the slab ends.

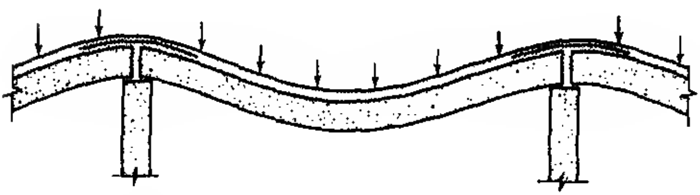
5.3 Composite Plank, Moment Continuity and Horizontal Shear

eavy loadings or fire considerations require greater flexural apacity than that of the most heavily prestressed section of a iven thickness. In such cases negative moment continuity at the upport can help to reduce the maximum positive bending moment. ests,[(122)] however, have indicated that continuity reinforcement laced in the grouted key joints and clamping action of wall anels provide only limited negative moment continuity.

..., wind loads, and partition loads (Fig. 44b). Loads due to the self-weight of the plank, which usually const at least 50% of the total load, are carried by simply supporte spans (Fig. 44a), unless the planks are shored while continuit established.



(a) Simply Supported Member



(b) Continuous Member

Fig. 44 Analysis Techniques for Floor Plank

Horizontal shear capacity at the plank-topping interface should assured by increased interface bond. Inadequate horizontal she capacity at the interface due to excessive shrinkage of the top can cause the topping to act independently, thus impairing the beneficial effect of composite action between the topping and t plank.

ribs. Where the edges of walls are chamfered more than ½ in the bearing length must be measured from the inside edge of the chamfer. Bearing lengths can be reduced from the values noted above if the bearing surfaces are armoured. However, short bearing length imposes tight and potentially costly manufacturing and erection tolerances.[123]

precast elements embedded in the joint. The function of con
to transfer forces from one element to another. If sufficie
and ductility is not developed in the connections, the avail
in the adjoining elements may not be fully utilized. As a r
design and detailing of connections and continuous joints is
factor in determining the safety and economic feasibility of
structures.[124,125]

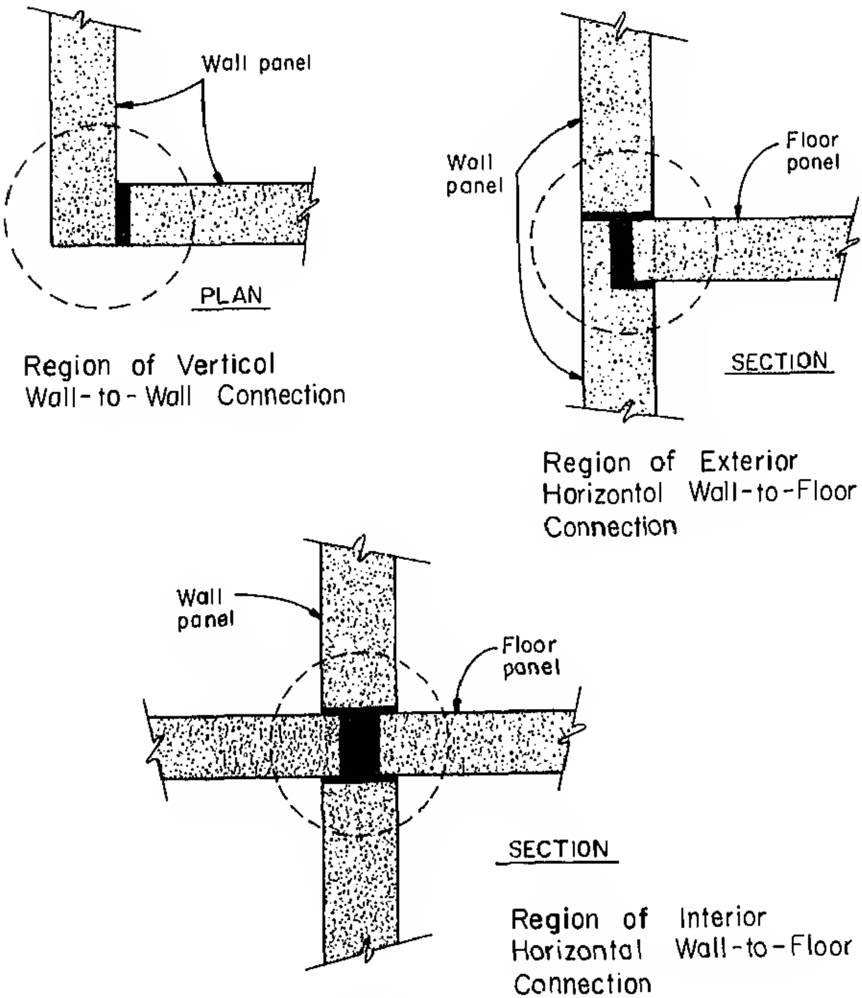


Fig. 45 C ction Regions

deflection, etc.;

(c) practical problems of casting and erection, including ances, etc.; and

(d) inaccessibility of joints which prevents inspection ar rectification.

Most procedures developed for connection design in panel structu of European origin, with European connections in mind. Therefor necessary to formulate complete and rational procedures and desi criteria for connections in American LP structures to include al factors affecting strength and behavior under normal and abnorma ing conditions. In the following text, strength and performance ments for the most widely used connection details are discussed provide a basis for analysis and design. A detailed discussion specific analysis and design methods, and suggested acceptance c will be given in a subsequent report.

Connections are classified with respect to location, direction, function, e.g., interior or peripheral, horizontal or vertical, wall-to-wall or wall-to-floor. The primary connections in an LF ture are divided into the following distinct groups (Fig. 46):

1. Interior horizontal wall-to-floor;
2. Exterior horizontal wall-to-floor;
3. Horizontal floor-to-floor; and
4. Vertical wall-to-wall.

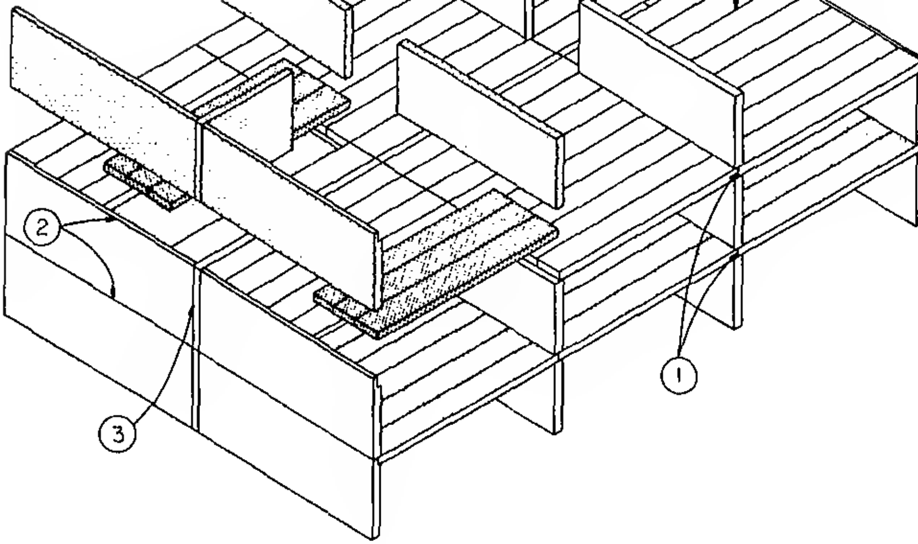


Fig. 46 Connection Classification and Location

e are also secondary joints, e.g., between lintels and wall panels, , however, since their design is usually straightforward, they will be discussed in this report.

2.6.1 <u>Interior Wall-to-Floor Connections</u>

This connection is also termed a horizontal function, a horizontal joint or a compression transfer connection. Various details are currently in use for this connection.[51-53,126-134] Most widely used among these is the "platform" connection (also known as the "closed" or "American" type connection) shown in Fig. 47.

In the typical "American" connection, prestressed hollow core slab extending over wall panels are continuously supported on bearing pads. Cast-in-place grout fills the vertical space between floor plank and usually a portion of the hollow cores of the plant itself. In the latter case paper or plastic dams are inserted into

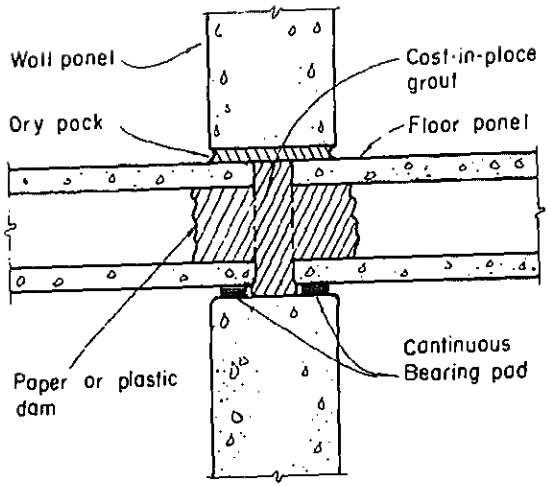


Fig. 47 Typical Interior Wall-to-Floor Connection

The function of the interior horizontal connection is to s
factorily transmit the forces described below and illustra
Fig. 48:

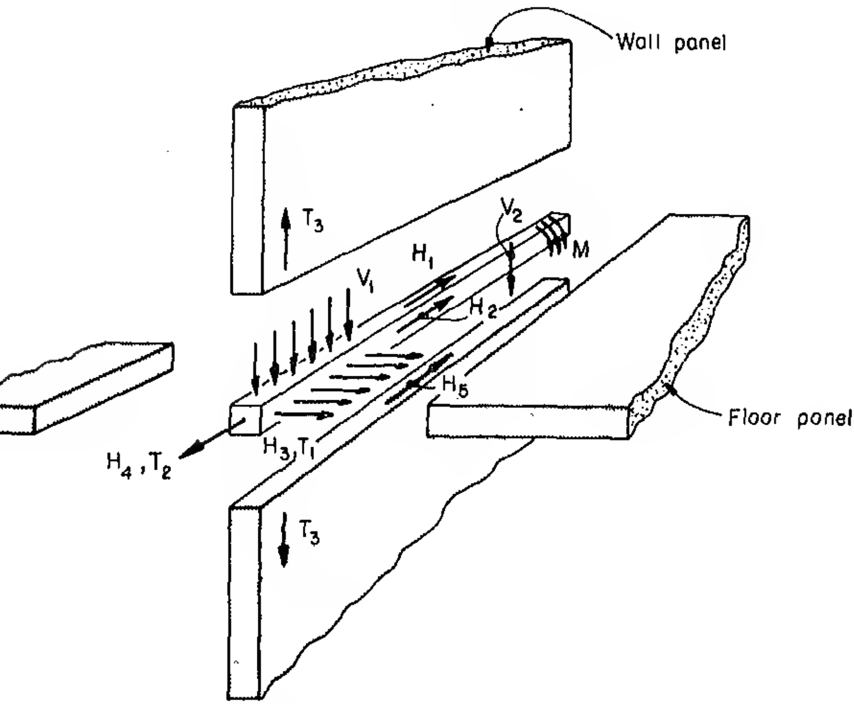
V_1 = vertical load from wall panel above;

V_2 = vertical load from floor loading;

H_1 = horizontal shear due to lateral loads on wa
this force also is present under abnormal

H_3, H_4 = horizontal thrust in the plane of the floor (tension or compression) resulting from diaphragm action and restraint force due to creep, shrinkage and temperature effects; and

M = transverse bending moment from floor continuity and load eccentricities.



. 48 Exploded View of an Interior Floor-to-Wall Connection and t
Various Connection Forces

fulfill the requirements of General Structural Integrity
escribed in previous sections, the connection must also satisfac-

action in the wall;

H_1 = horizontal beam shear force between the edges of wall panels and the joint as a result of integral cantilever or beam action (see Figs. 24 and 27); also present under normal loads;

H_5 = horizontal shearing forces to transfer the tie force T_2 into the panel below; and

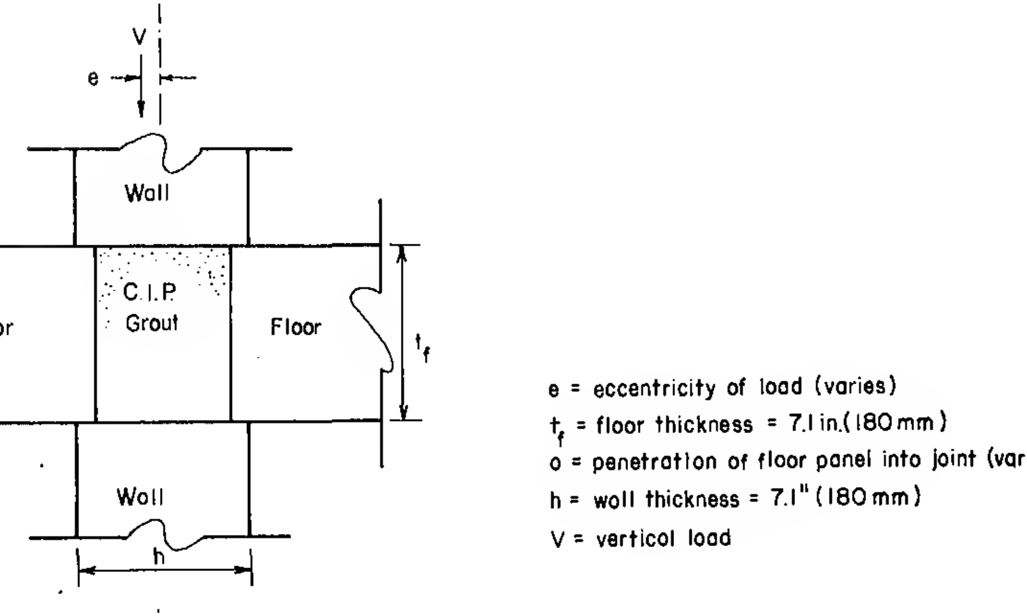
T_3 = vertical tie forces to provide suspension action and dowel action for developing H_1 and H_5.

2.6.1.1 Variables Affecting Connection Behavior

The connection is composed of various parts with forces acting concurrently. Consequently the state of stress in the connection is quite complex. In the last decade many significant studies of the behavior of connections have been reported.(104,122,125-127,129-138) The following is a summary of the results relevant to the "American" type connections.

Precast and Cast-in-Place Concrete Properties--Variations in the elastic moduli of the wall concrete, floor concrete, and the cast-in-place concrete appreciably affect the behavior of the connections.(91) Their relative values influence stress distribution within the connections and consequently the load-carrying capacity. Lugez(132) observed that separation of the floor panels from cast-in-place grout in the connections occurs under vertical compressive loading of very low magnitude. Thus, continuity between the floor panels and the

study concluded that tensile strain increases with reducing values of the elastic modulus of the cast-in-place grout, E_j.

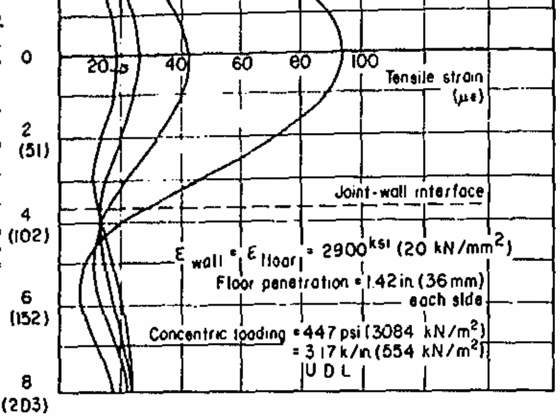


49 Platform Connection Details used in Ref. 91

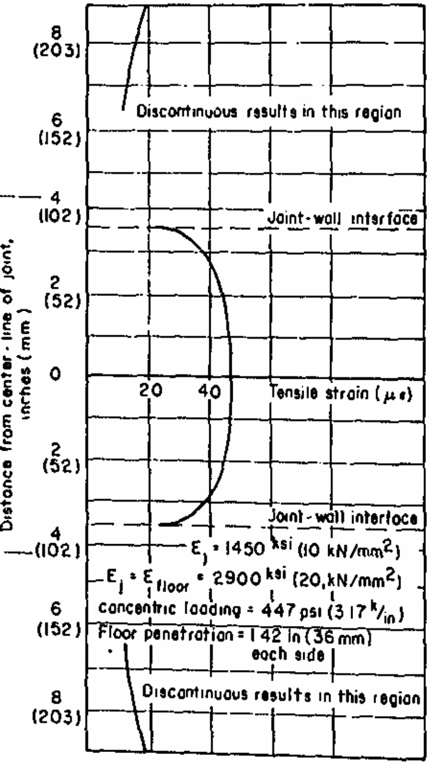
Depending on the values of the elastic moduli of the wall, floor and cast-in-place grout, a horizontal connection under concentric loading can fail in four basic modes, as shown in Fig. 52.

The connection fails as shown in Fig. 52a when concrete in the precast wall panel develops compression cracks near its end. This mode predominates if wall concrete is weaker than floor and cast-in-place grout.

When the hollow cores in the floor plank are left unfilled at the support, or when the floor plank is supported on "softer" material such as elastomeric or other bearing pads, the

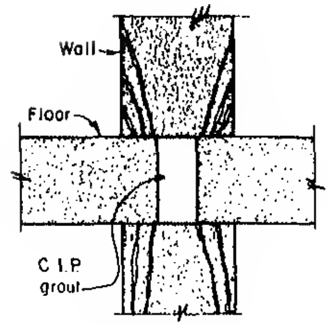


. 50 Tensile Strains along the Centerline in the Connection shown in Fig. 49 (Ref. 91)

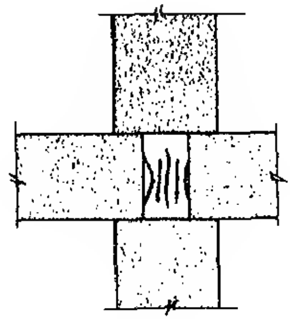


. 51 Tensile Strains along the Centerline of a Cracked Connection Shown in Fig. 49 (Ref. 91)

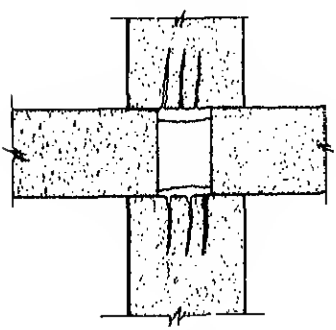
the floor plank is not eliminated by tensile forces, the
is in a biaxial state of stress and can fail in compressi
as illustrated in Fig. 52b.



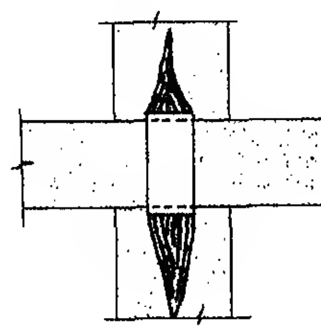
(a) Failure of Wall under Compression



(b) Failure of Cast
Grout



(c) Weak Cast-in-Place Grout and Strong Floor Concrete Cause Splitting of Wall Panel



(d) Failure through
of Strong Cast-
Grout into Wall

Fig. 52 Various Failure Modes of an Interior Wall-to-Floor Co

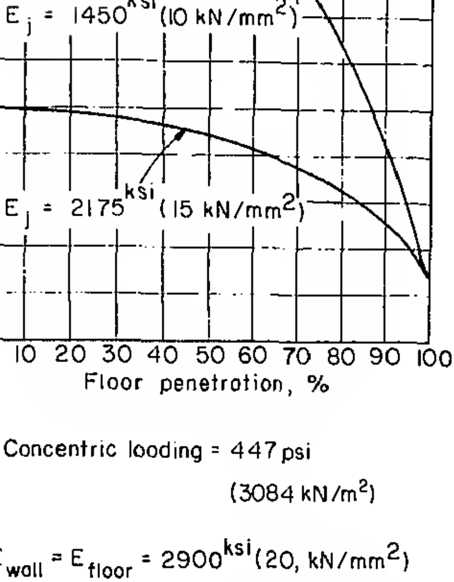
In the case where the cast-in-place grout is stiffer th in the wall panel and the floor plank, the cast-in-plac supports almost the entire load from the wall above. U these conditions, the cast-in-place grout wedges into t panel causing the splitting shown in Fig. 52d.

Experimental studies have shown that splitting of wall of major concern.(52,91,134) As a result capacity redu factors for reinforced concrete walls have been suggest Alternatively, test results have shown that splitting c minimized by use of proper transverse split-resisting forcement at wall panel ends.(134) Based on experimen tests related to American practice, definitive recomme will be presented in a subsequent report on connection

<u>Effects of Floor Penetration</u>--Floor penetration is def the measure of floor end-length bearing on the wall. penetration per unit wall thickness is termed the pene ratio, p, i.e.:

$$p = \frac{\text{sum of two floor panel end lengths bearing on}}{\text{wall thickness}}$$

Theoretical and experimental studies of solid slab sec carried out in England indicate that as the penetratic increases the tensile strain in the wall decreases.(91 the connection configuration shown in Fig. 49, this ef increases as E_j of the cast-in-place grout decreases (53).

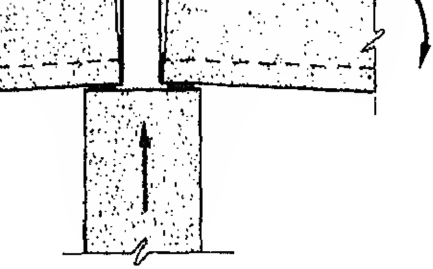


3 Effect of Floor Penetration on the Tensile Strains at the Center of the Connection in Fig. 49 (Ref. 91).

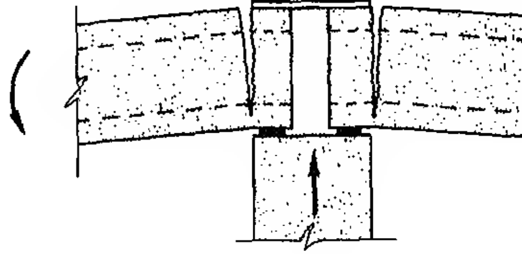
Floor Moment--Floor moment can be introduced in the connection if rotation of the floor panel at the joint is restricted by vertical clamping from the wall panel. Depending on the magnitude of clamping force and on the negative moment capacity of the plank, either of two modes of cracking exist:[122]

1. Separation at the interface between plank and cast-in-place grout in the connection can occur when the clamping force is insufficient to prevent plank rotation or when measures were taken to allow such rotation (Fig. 54a). Such rotation induces splitting forces in the wall above.

2. Alternatively, cracking in the plank near the support can occur when the induced moment exceeds the plank's moment capacity (Fig. 54b). This condition



rotates due to
ficient clamping
n at joint

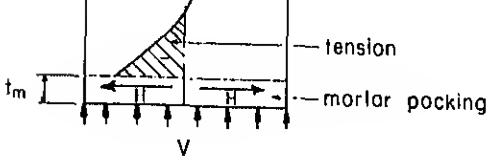


(b) Slab cracks due to insufficient flexural capacity

Cracking Modes at Interior Wall-to-Floor Connection due to Slab Moment

Load Eccentricities--The load carrying capacity of a connection can be reduced under load eccentricities. For a given eccentricity, the reduction in connection capacity depends upon various parameters: wall thickness, penetration ratio, and properties of cast-in-place and precast concretes.[132] Design charts are available to determine the reduction in capacity due to load eccentricities.[91,132]

Effect of Mortar Packing--After a wall is vertically adjusted, the space between the bottom of the wall and the floor slabs is filled with a drypack mortar. Workability, proper compaction and shape of drypack influence the capacity of the connection. Incomplete grouting or packing reduces the capacit of the connection substantially and greatly effects its overall behavior. If the drypack is poorly compacted at the center of



h = Wall thickness
t_m = Motor thickness
H = Horizontal force
V = Vertical force

Fig. 55 Theoretical Transverse Stress Distribution on a Wall E
(Ref. 132)

the connection, loads which would normally pass through the center grout will be transferred through the plank instead. This condition can cause premature splitting of the wall as shown in Fig. 52d.

If the stress-strain characteristics of the packing material is significantly different from that of the wall, the latera deformation of the mortar can generate horizontal forces at the bottom of the wall (Fig. 55). Finite element analysis h shown that if the mortar is of poor quality, or relatively thick, the magnitude of tensile stresses becomes sufficientl high to cause cracking and splitting of the wall end.[132] European experimental and theoretical studies indicate that the problem can be minimized if the following steps are taken:[52,132]

(a) the connection is properly made with well compacte mortar;

(b) the strength of the mortar is equal to or not less than 75% of the concrete strength of the wall; and

(c) the mortar thickness, t , is not greater than

considerable horizontal shear from the wall above to th
panel below under both normal and abnormal loading cond
If shear transfer mechanisms are not available, brittle
failures can occur, particularly under conditions prese
the damaged state.

When the connection is under compression, a portion of
shear can be transferred by friction between the precas
concrete wall panels and the connection. The magnitude
this shear force, S, is determined by the relationship:

$$S = \mu N \qquad \text{(Eq.}$$

where

N = compressive force normal to the direction interface; and

μ = coefficient of friction between wall pane mortar layer.*

If the magnitude of induced shear force exceeds the for
vertical reinforcing is necessary. The required vertic
reinforcing steel area, A_{vf}, can be conservatively calc
by using the shear friction analogy equation:[125]

$$A_{vf} = \frac{H}{f_y \tan \alpha} \qquad \text{(Eq.}$$

*In various specifications the value, μ, varies from 0.2 to 0.8.(

$\tan\alpha$ = coefficient of internal friction between precas wall and the drypack or grout, normally taken a 0.7.[59,104,139]

The simple shear-friction approach to designing joints has been adopted by the ACI code.[15] It assumes that the reinforcement across the crack acts as a tension tie rather than as a dowel. The cohesive strength of the concrete as well as the effect of compressive force across the connection are neglected. Since the interface inherently provides a convenient place for slippage to occur, the assumption to neglec the cohesive strength seems justified.

2.6.2 Exterior Wall-to-Floor Connection

This connection, which typically occurs only at bearing flank wall of the structure (Fig. 46), is also termed an exterior horizontal connection. Various details are currently in use.[52-4,102, 133,1 As in the case of the interior horizontal connection, the "America or "platform" type joint seems to be the most widely used (Fig. 56). In this detail the hollow core floor plank extends over the wall panels from one side only supported on bearing pads. The space between the plank and wall is filled with cast-in-place grou with paper or plastic dams inserted into the hollow cores to limit the extent of core filling. The void between the top of the plank and the bottom face of the wall panel is filled with drypack morta while the exterior face of the junction between the wall panels is caulked.

The forces and actions which exist in the exterior horizontal

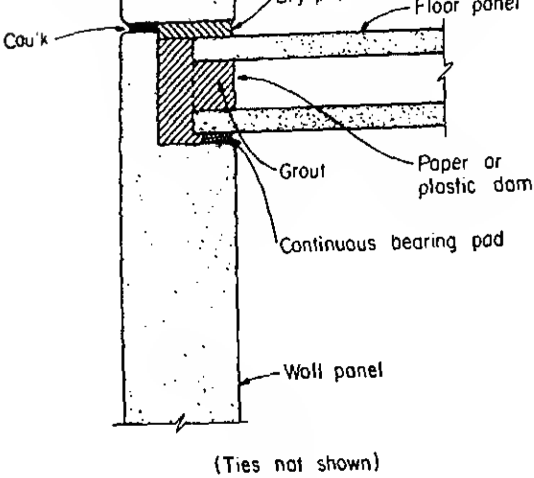


Fig. 56 Typical Exterior Wall-to-Floor Connection

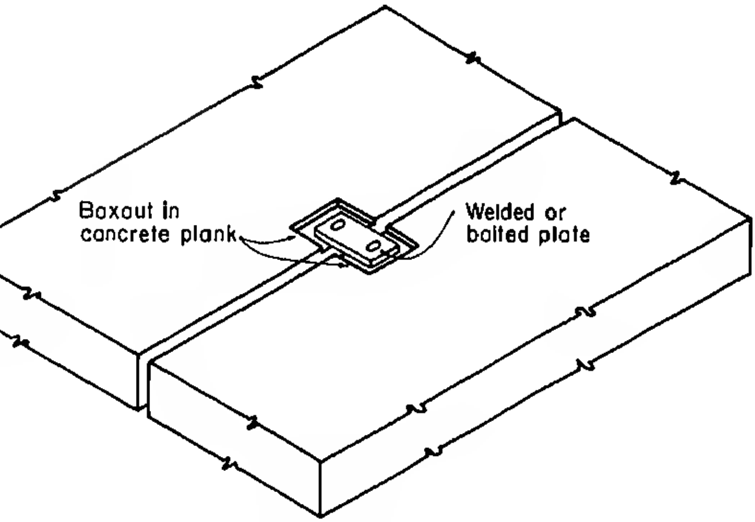
connection. The variables affecting its behavior are also si
to those discussed for the interior horizontal connection.
ever, the lack of symmetry in the exterior connection causes
eccentric stress distribution which reduces its load-bearing
capacity. Tests indicate that the ultimate stress for this
nection can be as low as $0.25f'_c$, as compared to $0.85f'_c$ for a
concentrically loaded wall panel.[140,142] In addition, the
eccentricities induced at the connection significantly affec
behavior and design of the exterior walls.

2.6.3 Longitudinal Connection between Floor Elements

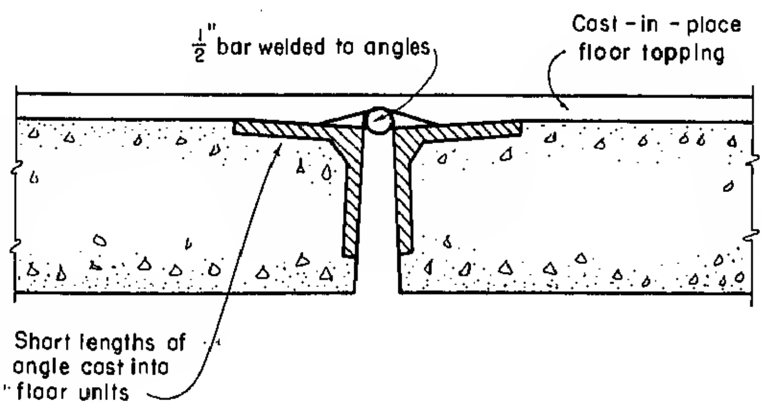
This connection, which occurs at adjacent vertical faces of
vidual floor elements, is usually termed a "shear key". As
cated in Figs. 57 and 58, various profile and reinforcement
are used for this connection.

indicates Cast - In - Place Grout

Fig. 57 Grouted Connections Between Floor Elements



Bolted Plate



Welded Bar

. 58 Mechanical Connectors Between Floor Elements

The joints introduce discontinuities which can adversely affect t floor's overall performance. To minimize such effects, the hori- zontal connection or shear key must resist with minimal deformati the following (Fig. 59):

V_3 = vertical (out-of-plane) shear induced from adjoini floor/roof elements subject to varying superimpose loads:

H_6 = horizontal (in-plane) shear resulting from the floor's diaphragm action; and

H_7 = normal force (tensile or compressive) resulting fr the floor's diaphragm action.

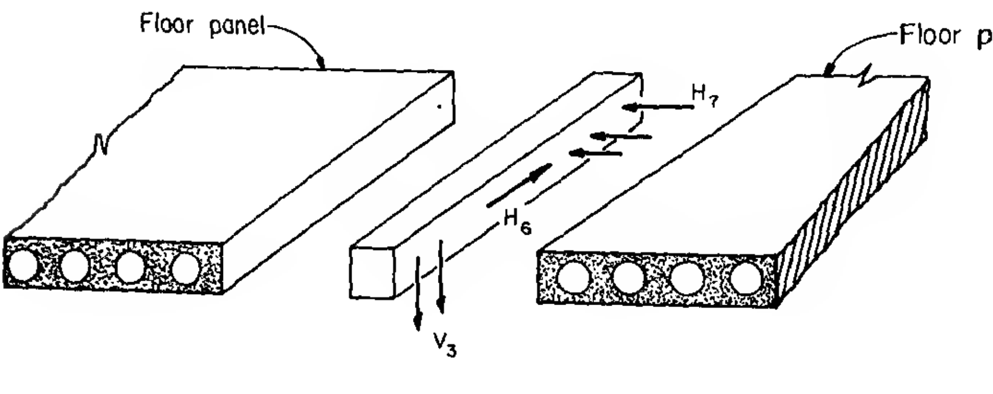


Fig. 59 Exploded View of the Horizontal Connection Between Floor Panels and the Various Connection Forces

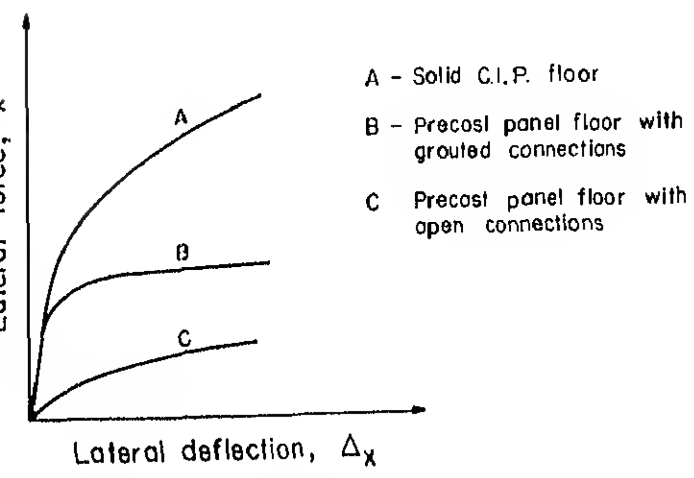
European type precast panels can act as effectively as a monolithic slab in horizontally stiffening a structure. The study determined the effects of longitudinal connections on the in-plane deformation of the following types of floors:

(a) solid cast-in-place;

(b) precast panels with grouted connections enclosed with a reinforced concrete ring beam; and

(c) precast panels with connections left ungrouted, enclosed with a reinforced concrete ring beam.

The load-deformation relationships of these floor systems ar shown in Fig. 60. The force-deflection curves illustrate that the ungrouted connections appreciably reduce the overal floor rigidity. However, the floor with grouted connections acts as effectively as a monolithic slab, until tensile cracking in the ring beam occurs.



ig. 60 Force-Deformation Relationships of European Types of Fl Systems under Horizontal Loading

the keys is approximately $3\sqrt{f_c'}$.(143) For typical bu configurations shear stresses developed in the horizon connection between adjacent plank should not exceed th value.

Based on the above discussion, two conditions should b filled to achieve diaphragm action:

1. The keyways between precast panels should b with grout or mechanically fastened togethe unless an adequate topping slab is employed design; and

2. Peripheral reinforcement should be adequate prevent the formation of tensile cracks und service load conditions.

2.6.3.2 Behavior under Vertical Shear

Vertical shear is induced in the keyway when adjacent are subjected to different vertical loads, e.g., one supports a concentrated load, such as a partition wal the adjacent panels carry only their own weight. The is essentially to deflect the adjoining panel edges, ensuring participation of such panels in the structur response.

Tests on slabs with grouted shear keys(143) show that ultimate vertical shear stress of the keys is approxi $3\sqrt{f_c'}$. Although shrinkage of the grout in the key ca a crack of 0.001 in. (.03 mm), such a crack width is

2.6.4 Vertical Wall-to-Wall Connection

This connection occurs at the vertical edges of adjoining w panels within interior or exterior wall assemblies. The va details used for this connection are classified as dry or w according to the methods and devices employed.

The vertical connections used in American practice are almo exclusively of the "dry" type, which increase the speed of tion. They consist mostly of steel plates or angles welde bolted to inserts cast within the panel (Fig. 61).

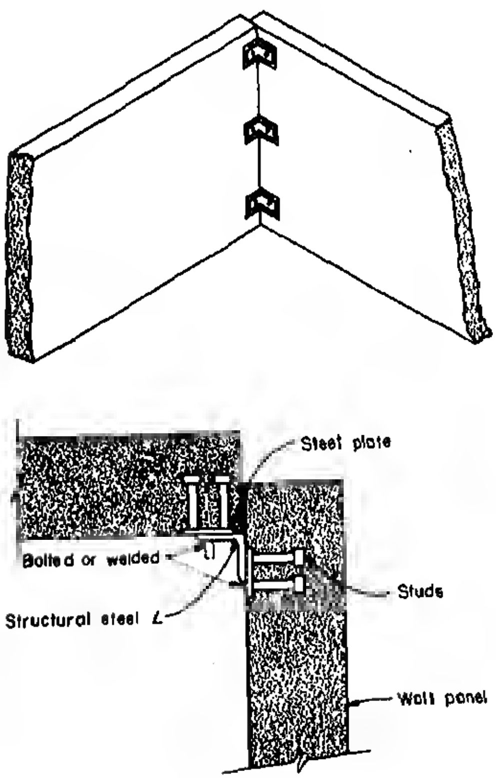


Fig. 61 Typical "Dry" Vertical Wall-to-Wall Connection

project into the connection grout. To optimize the
of "special" connections the reinforcing loops prot
panels are interlaced by one or more vertical reinf
before the grout is placed.

Under normal loadings the basic function of a verti
is to equalize deformations between adjoining stack
through in-plane shear from one stack to the next.
deformations can result from:

(a) different stress levels in the panels;
(b) different moduli of elasticity of the pan
(c) varying creep, shrinkage and temperature
panels.

When the vertical connection is located within a wa
subjected to lateral forces, shear (V_4), and compre
sile stresses (H_8) will be transferred through the
in Fig. 63.

Abnormal loading conditions significantly increase
shear requirements for the connection. This is due
lever or beam action that is developed in the wall
lower panel becomes ineffective. Under these condi
vertical connection must act as a support for the e
on the panel. As a result, the magnitude of shear
is specified for abnormal conditions will usually g
of the connection for shear.

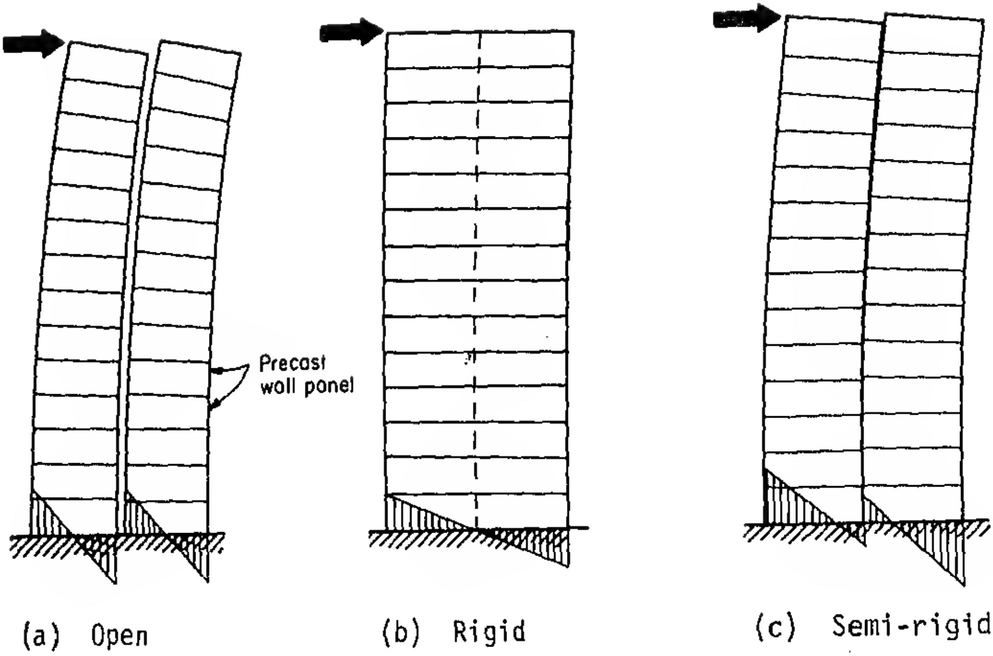


Fig. 64 Drift and Stress Distribution in Shear Wall Assemblies

Open Connection--With this type of connection no shear force is transmitted from one stack of panels to another. As a result, the vertical edges are free to slide past each other (Fig. 64a). In practice the open connection should be avoid since (a) it can cause slab cracking at the connection, and (b) slip at the vertical interface significantly reduces overall stiffness.

Rigid Connection--A vertical connection is called rigid when there are no relative displacements of the adjoining wall panels. A rigid connection provides full interaction betwee

Semi-rigid Connection--In practice some slip occurs in almost all vertical connections.[145,146] The shear rigidity of the semi-rigid connection is less than the rigidity of the stacks of adjoining wall panels. As a result, the shear stress transferred across the connection is accompanied by a degree of slip between the adjoining wall panels (Fig. 64c).

For dry joints the rigidity primarily depends on connector strength and type. The various connection types (i.e., bolte and welded), have different force-deformation characteristics that influence the degree of slip in the connection.

For wet connections the rigidity can be characterized by a rigidity modulus, G_j, defined as the ratio of an increment of shear stress divided by a corresponding increment of shear displacement. Factors affecting the rigidity of a wet connec tion include:

(a) material properties of cast-in-place grout and rein forcing steel;
(b) amount and distribution of reinforcing steel;
(c) connection width;
(d) shape and spacing of vertical keys (castellation); and
(e) height of the structural wall.

Slip can be minimized in either connection with proper attention given to the details. For purposes of analysis a nominal amount of slip is insignificant.

system of forces and moments, the connection can fail in one of the following modes:

(a) metal connector failure;

(b) insert pull-out failure; or

(c) concrete failure.

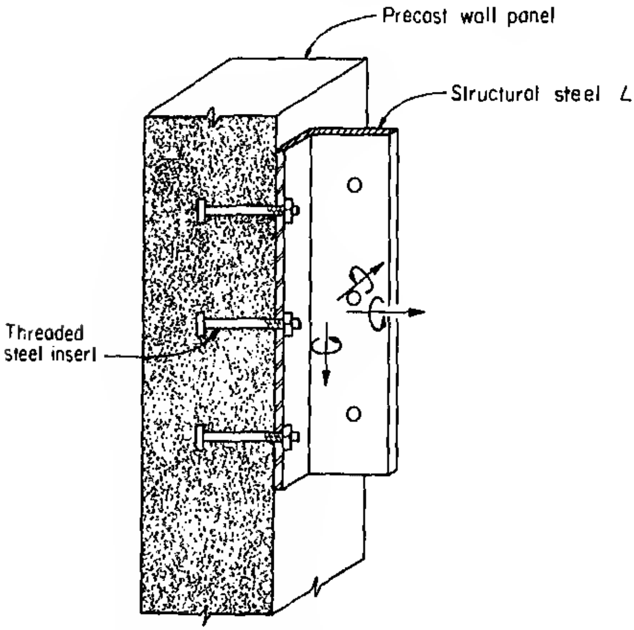


Fig. 65 Forces and Moments on a Dry Vertical Connection

Failure mode (a) occurs when the steel angle or insert yields. Information on the nature of such failure, as well as design procedure to prevent this failure mode, is readily available.[147-149] In mode (b), the entire insert can pull out of the concrete due to bond failure between concrete and

tension failure occurs in the form of a concrete cone having its apex slightly past the deepest part of the insert, as shown in Fig. 66. Diagonal tension failure takes place when the tensile strength of the "shear cone" surrounding the insert is less than the strength of the anchoring elements of the insert.[152] The results reported in Ref. 152 indicate that the embedment length of an insert is the most important factor in determining the pull-out capacity of the insert. Various design formulae to determine ultimate pull-out strength of studs and inserts are available.[113,152]

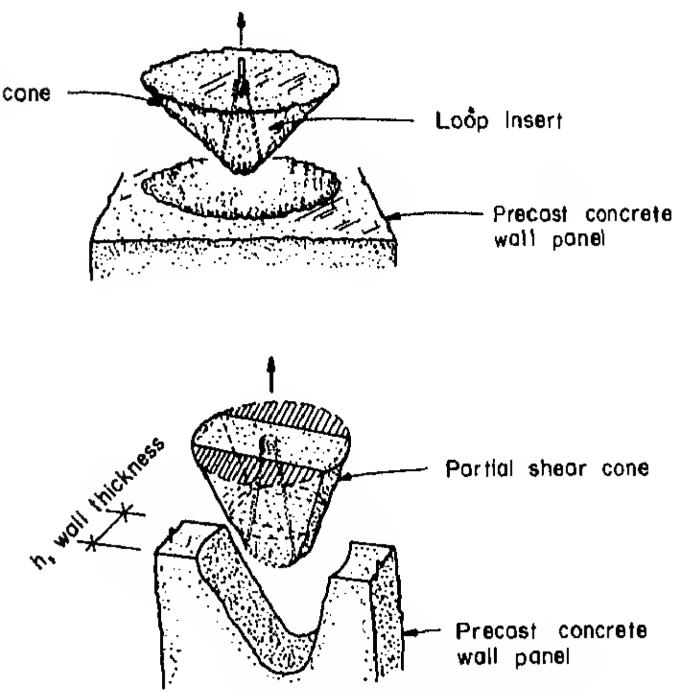


Fig. 66 Concrete Cone Failure at Ultimate Pullout Load

elastic modulus of the concrete. If adequate concrete bear area or strength is not available, bearing failure can occu Empirical formulae to calculate the allowable shear capacit of a stud bolt are readily available.[113]

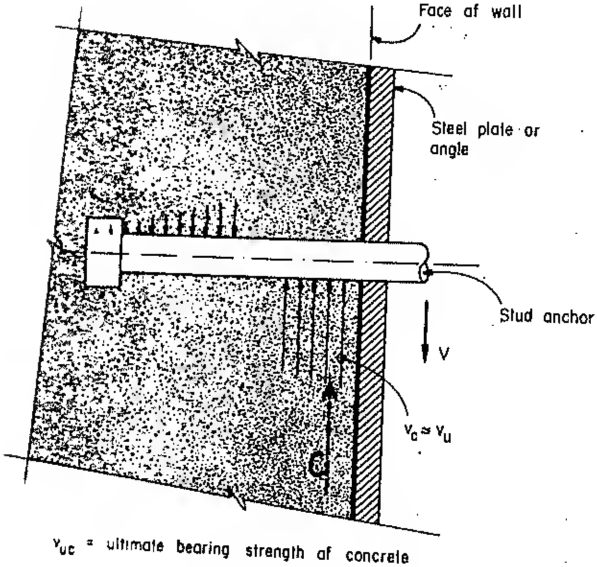


Fig. 67 Stress Distribution at Concrete Bearing Failure of Dry V Connection

2.6.4.3 Behavior of Wet Vertical Connections

Many notable European studies have been performed on the wet connection, and considerable material is available.[91,124, 133,135,137,145,146,153-155] A brief summary is presented below.

above 85 psi. Such connections are subject to a sudden brittle failure. Plane reinforced connections, on the other hand, are relatively ductile, and can develop considerable ultimate shear resistance[124] through dowel action and shear friction.

Grooved--Unreinforced grooved connections can exhibit a very high rigidity because the groove increases the bond surface. However, as in the case of unreinforced plane connections, such resistance is not dependable, due to loss of bond resulting from shrinkage. Reinforced grooved connections exhibit higher elastic resistance than reinforced plane connections, while their ultimate shear resistance is approximately the same.[53,91,124]

Keyed--Keyed connections are the most commonly used wet vertical connections. The elastic resistance against slip is provided by interlocking rather than by bond. Once the principal tensile stress in the connections exceeds the tensile resistance of the cast-in-place grout, cracks develop and shear resistance decreases. In a reinforced section after cracking, the shear force is transferred across the connection through diagonal strut action, with the compressive and tensile components of the strut action resisted by the concrete and steel respectively.[145] Empirical formulae to determine maximum shear resistance of keyed connections are available.(

tance to structural safety. Recognizing this, and considering the many uncertainties in connection design, various codes have recommended that connections be designed with higher safety factors than the adjoining elements. The higher safety factors may be further justified in light of the connection's susceptibility to inferior quality control during construction.

A_{vf} = area of vertical reinforcement

A_w = cross-sectional area of the wall

b = length of a precast wall panel

C = compressive force on concrete

C_u = ultimate load-carrying capacity of a connection compression

e = eccentricity of compressive loading on a wall fr centroidal plane of the wall

E_{floor} = elastic modulus of concrete in floor panels

E_j = elastic modulus of cast-in-place grout in connec

E_{wall} = elastic modulus of concrete in wall panel

f'_c = compressive strength of concrete

f'_{cf} = compressive strength of concrete in precast floo

f'_{cj} = compressive strength of cast-in-place grout in c

f'_{cw} = compressive strength of concrete in precast wall

f'_m = compressive strength of mortar

f_s = stress in steel

f_y = yield strength of reinforcing steel

G_j = rigidity modulus (shear modulus) defined as the an increment of shear stress divided by a corres increment of shear displacement

GSI = general structural integrity

h = precast wall panel thickness

H = tangential shear along an interface

ℓ = height of precast wall panel

ρ	=	penetration ratio = $\dfrac{\text{sum of two floor panel end lengths bearing o}}{\text{wall thickness}}$
p	=	ultimate pull-out capacity of stud or insert as g by concrete
S	=	shear force
SF	=	safety factor of design
t_f	=	thickness of floor panel
U	=	required strength to resist design loads
v_{uc}	=	ultimate bearing strength of concrete
α	=	angle of internal friction between precast panel mortar layer
μ	=	coefficient of friction between wall panel and mo layer
γ	=	load factor

codes and design practices.

eptable risk: That level of risk deemed toler

idental eccentricity: An eccentricity which exists as direct result of errors in eith manufacturing or erection proce

ual risk: See "risk".

ual size: A size found by measurement.

ernate path: The substitute load flow patter taken after an abnormal loading caused the "ineffectiveness" of primary load-bearing element.

embly: An aggregate of panels.

ring capacity: The maximum unit pressure which material will withstand without failure or without settlement t amount detrimental to the integ or the function of the structur

ttle connection: A connection which depends on f tion and bond under compressive loadings and consequently exhib little or no ductile characteri

learance:	Distance considered in the calcula... of a work size in order to achieve
onnection:	A position or region where two or more building components, panels o assemblies are put together or united.
ontinuity:	The capacity for load transfer between two or more elements where load is axial, shear, moment, or a combination thereof.
oordinating dimension:	Dimension of the spatial envelope containing the precast panel.
ross-wall system:	A large panel system in which the load-bearing walls are perpendicul to the longitudinal axis of the building.
flection index:	Ratio of the lateral deflection to the building height.
eformation:	A change in dimension or shape due stress.
viation:	The difference between a size or position (actual, limit, etc.) and specified size or position.

confined area; also, the mixture s
placed.

acked concrete: A concrete mixture sufficiently d
to be consolidated by heavy rammi

ility: The measure of a structural compo-
nent's (element or connection) ab
to sustain inelastic deformations
i.e., the ratio of the maximum de
mation to the yield deformation.

ent vulnerability: The susceptibility of a structura
element to the effects of an abno
load due to its location within t
structure.

ure: A loading or deformation state be
which a connection, member or str
ceases to fulfill its function.

k wall assembly: An assembly of wall panels along
periphery of the building.

r panel: A horizontal precast concrete ele
reinforced with mild or high-stre
steel.

r plank: A horizontal precast concrete ele
typically extruded and reinforced
with high-strength steel. Also k

	to inhibit progressive collapse wh retaining structural stability; achieved through a degree of conti combined with a degree of ductilit of the components and connections a structure.
ut:	Mixture of cementitious material a aggregate to which sufficient wate is added to produce pouring consis without segregation of the constit
uting:	The process of filling with grout.
uced deviation:	Deviation induced by human error a the limits imposed by measuring instruments.
ffective element:	An element which has been adversel affected by an abnormal load so th it no longer functions as a load-bearing member to the extent origi designed.
erent deviations:	Deviations in shape and position c panel in service.
egral column:	A column located within a wall par which can function as a vertical support under abnormal conditions.

such units.

e panel structure: A structural system composed of vertical load-carrying elements of large precast wall panels with pre-cast floors and roofs of panels or planks.

-bearing wall: A wall designed and built to carry superimposed vertical and shear loads, as opposed to non-load-beari walls.

factor: The ratio of the ultimate collapse load to the working load in a struc or section.

flow: The path taken by a load through th structural assembly from its point application to its point of resist-ance (foundation).

gitudinal tie: The horizontal tie used to link adjacent spans of floor elements.

hanical bond: The bond attributed to keying or interlocking rather than adhesion.

ed-wall system: A large panel system composed of a combination of cross-wall and spin s stems.

lotional removal: The approach used to establish in rity within a structure in which element or portion of the structu is assumed totally removed.

Partial stability: Stability of the structure in the damaged state.

Penetration ratio: Defined as the ratio of the sum two floor panel bearing end leng to the wall thickness; used for interior horizontal connections.

Peripheral tie: The horizontal tie used to creat ring beam at the exterior of eac story.

Progressive collapse: The phenomenon in which the spre an initial local failure eventua results in the collapse of an en building or a disproportionately large part of it.

Recipient panel: A floor panel which is subject debris loading.

Return wall: A vertical element used as a co tion of a load-bearing wall ele at a right angle to it.

unfactored normal loading conditio

pecified dimension: Dimension required for panel fabri tion so that the panel, when asser will comply with spatial disciplin

pine-wall system: A large panel system in which the load-bearing walls are parallel t the longitudinal axis of the buil

tructural eccentricity: An eccentricity which exists as a direct result of a structural det

olerance: The allowable deviation from a specified dimension.

raditional loading condition: See "Normal loading condition".

ransverse tie: The horizontal tie used to latera link adjacent wall elements in th same vertical plane.

ltimate load: The maximum load which may be pla on a connection, member or a stru ture before its failure; also, th load at which a connection unit o structure fails.

ertical tie: The tie used to vertically link adjacent lifts of wall elements the same plane.

either load-bearing or non-load-
bearing; usually one-story in he
with lengths typically ranging f
10' to 45'.

Warping: A deviation of a floor or wall p
surface from its original shape,
usually caused by temperature or
moisture differentials within th
slab or wall.

resolve the problems associated with abnormal loadings. However, th
primary initiative and responsibility for defining, quantifying, and
guiding belongs to those regulatory agencies charged with drafting c
of practice and building regulations. In attempting to prepare crit
to reduce the level of risk from abnormal loadings, recent codes thr
out the world have taken one or a combination of the following appro

(a) the "cautionary note" which warns the design engineer of t problems;

(b) the "alternate path" which requires the engineer to explic consider removal of any structural element;

(c) the "specified abnormal loading" which requires the engine to design against an arbitrary overpressure load; and

(d) the "general structural integrity" which attempts to secur adequate integrity through minimum detailing practice by establishing a degree of continuity and ductility within a between structural elements.

For purposes of illustration and comparison, the methods used by var countries are described below.

.1 British

Reaction to Ronan Point was immediate. Six months after the disaste the British Government issued regulations intended to alleviate the problem of progressive collapse.(156) These regulations, were adopt n 1970 as the 5th amendment to the 1965 Building Regulations.(157) ode provisions, which applied to structures over five stories, requ hat a building be designed by the engineer so that a failure result rom the removal of any portion of any one structural member be loca ithin a story and be limited to three stories. A local failure wit story was defined to be 750 square feet or 15% of the floor area.

problem to the engineer.

For economic considerations and for practicality, this hastily dev solution came under much criticism.(158-166) As a result, these r tions received considerable attention from code-writing bodies and significantly modified. Under present British specifications,(59) certain minimum details for continuity and ductility are incorpora into the design, the 5 psi or member removal design are not requir

C.2 French

The French (CEB) regulations(54) were among the first codes to rec the problems of progressive collapse and abnormal loads in LP buil Within the introduction there is an immediate cautionary note whic states: "One can hardly overemphasize the absolute necessity of ef tively joining the various components of the structure together to obviate any possible tendency for it to behave like a 'house of ca and of organizing it accordingly." Although the regulations also contain further minimum detailing specifications, they are not con sidered as thorough as those contained in the British Standard.

The French reaction to the Ronan Point disaster was not as volatil that of the British. It was argued that the CEB regulations alrea recognized the problems of continuity and ductility in LP building furthermore, it was noted that the French regulations for gas inst tions were extremely restrictive, therefore gas explosions in high blocks were considered less likely.(167,168) Their first point ha well taken, however, regarding the latter, elimination of the occu of one abnormal loading condition is arbitrary considering the man other abnormal events that can occur.

. Explosions are only one example of so-called excessive load-
ings, all of which have to be considered.

. All kinds of buildings, irrespective of construction type anc
size (height), should be considered.

. Stability must be considered on the large scale (the total
stability of the buildings), and in the details (the keeping
of the separate parts together).

. It is of greater practical importance to limit the scale of
the damage than to try to eliminate damage totally.

. Excessive loadings can be of different magnitudes, and it is
desirable to apply precautions that are balanced from a
statistical point of view. Excessive loadings are more
important in bigger buildings than in smaller ones.

edish Regulation draft[82] which has now been adopted[96] was
on the above recommendations combined with additional background
s.[78,80,86,98] It emphasizes the importance of sufficient
th and ductility in all connections of a framework. The Swedish
llows excessive loadings to be taken into account in alternate
by localizing the damage; by designing with sufficient strength
id damage; or by decreasing the risk of occurrence of an excessi
The main interest, however, is centered on specifications to
ze damage and resist progressive collapse by ensuring that both
e forces and the connection design are adequate in size and
y. The regulations are similar in nature and intent to those of
itish Standard.

ntegrity, strength ...
rogressive collapse due to local failure are reduced to a level comme
rate with good engineering practice.

though there is a substantial commentary[83] to the code provision,
he engineer is provided only a qualitative prescription for solving
roblem. The code offers no minimum detailing practice to establish
uch integrity and thus places this burden on the design engineer.

5 United States

n the United States there have been two cases where criteria to deal
ith progressive collapse were adopted. The first was the Guide Cri-
eria[169] used in the evaluation of housing systems for the Operatio
reakthrough program. The second[170] was prepared by the Federal
ousing Administration of HUD for use in preparation of Structural
ngineering Bulletins for those industrialized systems being develope
nd built with FHA mortgage guarantees. Both documents were adaptati
f the early British regulations and required either the alternate pa
ethod or the specified abnormal load approach (5 psi). As was the c
n Great Britain, these early U.S. regulations immediately came under
ttack from the profession as being too radical, uneconomical, and
nappropriate. As a result, the 1971 HUD document was revised a numb
f times, with the current revision[171] issued in May 1974, requirir
ominal diaphragm and wall ties, and also, either a 5 psi overpressur
esign, or a notional removal design.

ontrary to the approaches used in the above documents, the PCI Com-
ittee on Precast Concrete Bearing Wall Buildings, has developed a dr
anual[53] which presents design recommendations emphasizing the inte
ependence of elements and the need to provide continuity and ductil-
o establish a single coherent assembly. Much of the minimum sug est

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To convert from	to	multiply by
Length		
inch (in.)	centimeter (cm.)	2.54
inch (in.)	meter (m.)	0.0254
foot (ft.)	meter (m.)	0.3048
yard (yd.)	meter (m.)	0.9144
Area		
square foot (sq.ft.)	square meter (sq.m.)	0.0929
square inch (sq.in.)	square centimeter (sq.cm.)	6.452
square inch (sq.in.)	square meter (sq.m.)	0.0006
square yard (sq.yd.)	square meter (sq.m.)	0.8361
Volume		
cubic inch (cu.in.)	cubic centimeter (cu.cm.)	16.39
cubic inch (cu.in.)	cubic meter (cu.m.)	0.0000
cubic foot (cu.ft.)	cubic meter (cu.m.)	0.0283
cubic yard (cu.yd.)	cubic meter (cu.m.)	0.7646
gallon (gal.) Can. liquid**	liter	4.546
gallon (gal.) Can. liquid**	cubic meter (cu.m.)	0.0045
gallon (gal.) U.S. liquid**	liter	3.785
gallon (gal.) U.S. liquid**	cubic meter (cu.m.)	0.0037
Force		
kip	kilogram (kgf)	453.6
kip	newton (N)	4,448.
pound (lb.)	kilogram (kgf)	0.4536
pound (lb.)	newton (N)	4.448
Pressure or Stress		
kip per square inch (ksi)	kilogram per square centimeter (kg/sq.cm.)	70.31
pound per square foot (psf)	kilogram per square meter (kg/sq.m.)	4.882
pound (force) per square foot (psf)	pascal (Pa.)†	47.80
pound per square inch (psi)	kilogram per square centimeter (kg/sq.cm.)	0.0703
pound (force) per square inch (psi)	pascal (Pa.)†	6,895.
Mass (Weight)		
pound (lb.) avdp.	kilogram (kg)	0.4536
ton, 2,000 lb.	kilogram (kg)	907.2
grain	kilogram (kg)	0.0000
Mass (weight) per Length		
kip per linear foot (klf)	kilogram per meter (kg/m.)	0.0014
pound per linear foot (plf)	kilogram per meter (kg/m.)	1.488
Mass per Volume (Density)		
pound per cubic foot (pcf)	kilogram per cubic meter (kg/cu.m.)	1
pound per cubic yard (pcy)	kilogram per cubic meter (kg/cu.m.)	
Temperature		
degree Fahrenheit (deg. F.)	degree Celsius (C)	$t_C = (t_F$
degree Fahrenheit (deg. F.)	degree kelvin (K)	$t_K = (t_F +$
Energy		
British thermal unit (Btu)	joule (J)	1,056
kilowatt-hour (kwh)	joule (J)	3,600,000
Power		
horsepower (hp) 550 ft.-lb./sec.	watt (W)	
Velocity		
mile per hour (mph)	kilometer per hour	
mile per hour (mph)	meter per second (m./s.)	